CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

JULY 1960 **VOL. LV. NO. 7**

FIFTY-FIFTH YEAR OF PUBLICATION

PRICE 2s. ANNUAL SUBSCRIPTION 24s. POST FREE. $3.90 in Canada and U.S.A.

LEADING CONTENTS

No. 632 ISSUED MONTHLY *Registered for Canadian Magazine Post*

INDEX OF ADVERTISERS

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Volume LV, No. 7. LONDON, JULY, 1960.

EDITORIAL NOTES

Models as Aids to Design.

OF all the aids available to the designer of structural and civil engineering works, models are the most realistic. Mathematical theories are applicable only as far as the assumptions that must be made to render the mathematics workable comply with reality. Calculators, whether manual, mechanical, or electronic, can operate only on numerical data derived from theories. Generally, however, models can simulate most of the indeterminable factors of a complex structure, and are used for proving or disproving results obtained from theoretical considerations ; they are also used in many cases where the action of a structure is so complex that analytical solutions are impracticable or even impossible. Models may be simple planar representations of a structural frame or the like as have been used for many years in teaching and in practical design to establish deformations, as measured by slopes and deflections, from which bending moments and shearing forces can be derived. More recently three-dimensional models of structures such as bridges, dams, silos, and thin-slab roofs have been used with success. Indeed, it has been stated by the Ministry of Public Works of Portugal that the design of dams in that country is based entirely on the study of models and that confirmation by calculation is deemed unnecessary ; that is to say, the experimental method is accepted as completely reliable. Models are more necessary now that concrete is being used in forms unlike elemental timber or steel construction. One of the first models of a large concrete structure is said to be a geodetic-frame roof of a hangar constructed in Italy in 1935.

A three-dimensional model automatically reproduces boundary conditions which frequently are indefinable, but if a model is to be entirely satisfactory it must simulate not only the shape of the structure but the properties of the materials of which the structure is made. Complete simulation is not easy, especially of reinforced concrete structures, and much depends on the purposes for which the tests are made. If it is necessary to examine only the behaviour of the structure when it is subjected to working loads, that is while it is in an elastic state, then the material of which the model is made need not be the same as that of the structure so long as it is elastic, homogeneous, and isotropic, and conforms to Hooke's law. Plastic materials are frequently used in such cases, and have the additional advantages of cheapness, speed of making the model, and ease of machining, welding, and the attachment of gauges.

If a model is to be tested to destruction, as is the case when the ultimate load or the primary weaknesses are to be determined, then the material must be one that behaves from a state of no stress to failure exactly like the material of the actual structure or as much like that material as is possible as regards its characteristics of resistance and rheology. Concrete is the most suitable material for a model of a reinforced concrete structure. A concrete, which is giving very satisfactory results in Italy, consists of cement mortar and pumice aggregate and is used for tests at the elastic stage as well as at failure.

The testing of models, as opposed to structures of full size, is further complicated by the necessity to co-ordinate the scale of various factors such as linear dimensions, forces, time, temperature, and acceleration. The time during which a model is loaded or for which a load is maintained also affects the results because of the " viscosity " of most structural materials, that is the measure of the rate at which a deformation or stress is transmitted through the material from one part of the structure to another. Time cannot be reduced in scale ; therefore a factor must be introduced into the results to allow for the ratio between the period of action of the loading on the model compared with that on the actual structure. When examining the behaviour of a structure under dynamic influences such as earthquakes, acceleration must be considered, and the results are complicated since the scales of acceleration due to gravity, like those of time and temperature, cannot be manipulated. Much research has been directed to the determination of the effect of the different scales of these factors. The difficulties arising from the problem of scale are largely overcome if tests on full size structures are undertaken. The cost of such an investigation may sometimes be prohibitive, but there are cases in which it has been worth while to test an element of a structure containing numerous identical elements ; an instance is the hyperbolic lobes forming the roof of the stands at Madrid racecourse.

Some of the statements in the foregoing are from papers read at a symposium on Models for Structural Design held last year in London by the Cement and Concrete Association. One of the speakers quoted Michelangelo as saying that " a builder's most blessed expense is the money spent on models ".

In view of the many factors that are involved in the making and testing of a model structure, the model must represent the actual structure so far as practicable, and the engineer must have confidence in the accuracy of the loading and measuring apparatus. The loads and pressures on an actual structure can be reproduced fairly accurately ; for example, the uniformly-distributed load on a floor or roof can be simulated by imposing on, or suspending from, the model a large number of concentrated loads. Hydrostatic pressures, as in the case of models of dams, can be represented by horizontal loads imposed by numerous jacks. Measuring instruments are of two principal types, deflectometers and strain-gauges. It is common experience that the results obtained from strain-gauges applied to concrete can be very erratic, and consequently it is necessary that the results be interpreted by a competent investigator ; also, if the results are to be reliable the models must be made and tested by thoroughly experienced people. Good models properly tested are not cheap, but the results obtained may effect much saving in the cost of a structure, especially if it be of such a form that its design by calculation would require a considerable amount of guesswork.

Some models used recently in Great Britain and elsewhere are illustrated and described on pages 265 and 283 to 286.

Characteristics of Symmetrical Segmental Members.

By A. BANNISTER, M.C., B.Sc., A.M.I.C.E.

THE analysis, by method of moment distribution, of frames containing segmental members has been considered in this journal, and in the following are given data by means of which the arithmetical processes can be facilitated. In this journal for November, 1957, Mr. A. Chronowicz gives three examples of the analysis of frames comprising curved or segmental members in which the constants for the members are determined by the method of column analogy. In this journal for March, 1959, Mr. T. A. Barta gives the constants, also by column-analogy methods, for a segmental member having a uniform second moment of area; the constants are in terms of ϕ which is half the angle subtended by the member and are developed as power series. Expressions are derived for the stiffness and carry-over factors, and the bending moments and forces at the ends of such members due to the displacements. The basic formulæ for these factors are given by Dr. E. Lightfoot.[1]

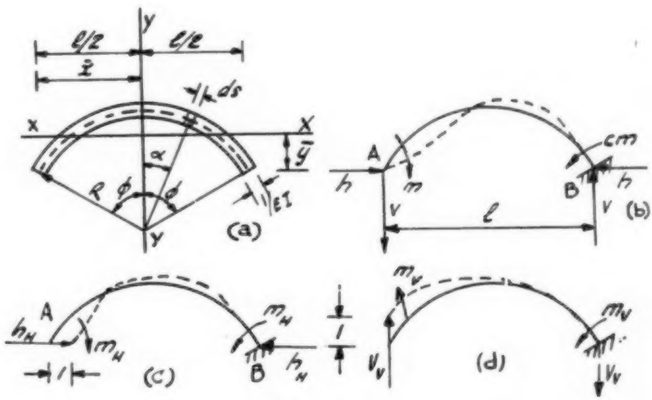


Fig. 1.

In this article charts are given to illustrate the relationship of some of these factors with ϕ. *Fig.* 1(a) shows the plan of the analogous column of a symmetrical curved beam of constant flexural rigidity EI and radius R. It can be shown readily that $y = R(\cos\alpha - \cos\phi)$, the area A of the column is $\frac{2R\phi}{EI}$, and $\bar{y} = R\frac{(\sin\phi - \phi\cos\phi)}{\phi}$. Therefore

$$I_{xx} = \frac{R^3}{EI}\left(\phi + \sin\phi\cos\phi - \frac{2\sin^2\phi}{\phi}\right);\ I_{yy} = \frac{R^3}{EI}(\phi - \sin\phi\cos\phi).$$

1.—E. Lightfoot. Generalized Slope Deflection and Moment Distribution Methods for the Analysis of Viaducts and Multi-Bay Ridged Portal Frames. "The Structural Engineer", January, 1958.

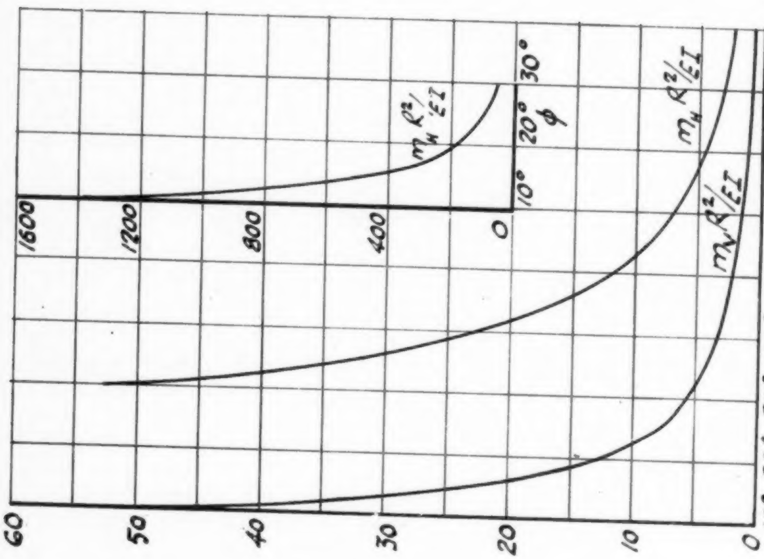


Fig. 3.—Values of $m_H\frac{R^2}{EI}$ and $m_V\frac{R^2}{EI}$.

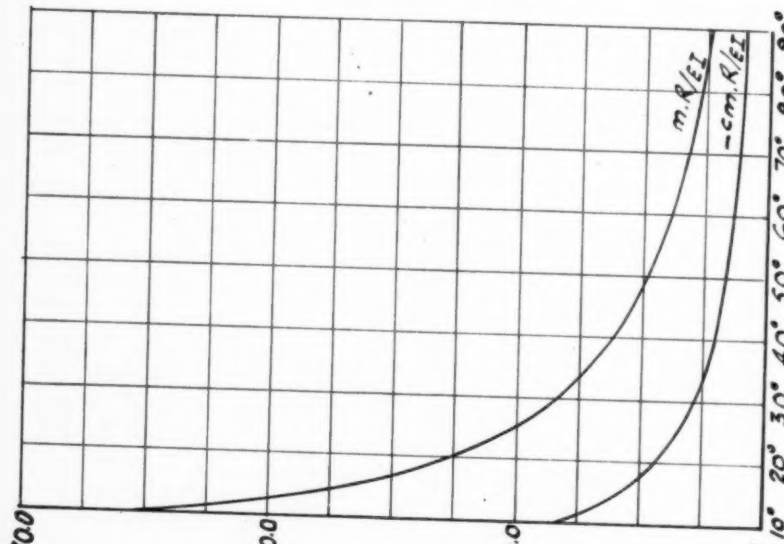


Fig. 2.—Values of $m\frac{R}{EI}$ and $-cm\frac{R}{EI}$.

The stiffness of a member is defined as the moment m required to produce unit rotation at A when B is fixed (*Fig.* 1), and it can be shown[1] that,

$$m = \frac{1}{A} + \frac{\bar{y}^2}{I_{xx}} + \frac{\bar{x}^2}{I_{yy}},$$

in which the terms A, $\bar{y}$, $\bar{x}$, I_{xx} and I_{yy} refer to the analogous column. Thus

$$m = \frac{EI}{R}\left[\frac{I}{2\phi} + \frac{(\sin\phi - \phi\cos\phi)^2}{\phi(\phi^2 + \phi\sin\phi\cos\phi - 2\sin^2\phi)} + \frac{\sin^2\phi}{(\phi - \sin\phi\cos\phi)}\right]$$

The corresponding value for a straight member of length L and constant flexural rigidity is $\frac{4EI}{L}$.

Also $h = \frac{\bar{y}}{I_{xx}}$ and $v = \frac{\bar{x}}{I_{yy}}$, their corresponding actions due to m being as shown in *Fig.* 1(b). By taking moments about B, the moment cm induced at that end due to the rotation at A may be determined. Assume first that cm acts in the same direction as m. Then $m - lv + cm = 0$, from which

$$cm = -m + lv = -m + \frac{lx}{I_{yy}}.$$

Therefore

$$cm = -\left(\frac{1}{A} + \frac{\bar{y}^2}{I_{xx}} + \frac{\bar{x}^2}{I_{yy}} - \frac{l\bar{x}}{I_{yy}}\right) = -\left(\frac{1}{A} + \frac{\bar{y}^2}{I_{xx}} - R^2\frac{\sin^2\phi}{I_{yy}}\right).$$

The factor c is the carry-over factor from A to B and, for a symmetrical member, from B to A. The curves in *Fig.* 2 show the variation of m and cm for values of ϕ between 10 deg. and 90 deg., and are plotted from the values given in *Table* I, which have been calculated from seven-figure mathematical tables.

TABLE I.

$\phi°$	$\bar{y}$	$\frac{mR}{EI}$	$-\frac{cmR}{EI}$	$\frac{m_H R^2}{EI}$	$\frac{m_v R^2}{EI}$
10	0·010R	25·40	8·28	1380·30	49·29
20	0·040R	12·78	4·33	177·50	12·36
30	0·089R	8·42	2·90	52·90	5·52
40	0·155R	6·21	2·19	22·50	3·12
50	0·235R	4·86	1·77	11·65	2·02
60	0·327R	3·94	1·49	6·84	1·41
70	0·427R	3·26	1·30	4·38	1·04
80	0·532R	2·74	1·15	2·99	0·80
90	0·637R	2·32	1·04	2·14	0·64

The effects of introducing unit horizontal and vertical displacements at A only are shown in *Fig.* 1(c) and 1(d) respectively. The directions of the forces and bending moments are those given by the applied displacements. It can be shown[1] that

$$m_H = \frac{\bar{y}}{I_{xx}} = \frac{EI}{R^2}\cdot\frac{\sin\phi - \phi\cos\phi}{\phi^2 + \phi\sin\phi\cos\phi - 2\sin^2\phi} = h.$$

$$h_H = \frac{1}{I_{xx}} = \frac{EI}{R^3}\cdot\frac{\phi}{\phi^2 + \phi\sin\phi\cos\phi - 2\sin^2\phi}.$$

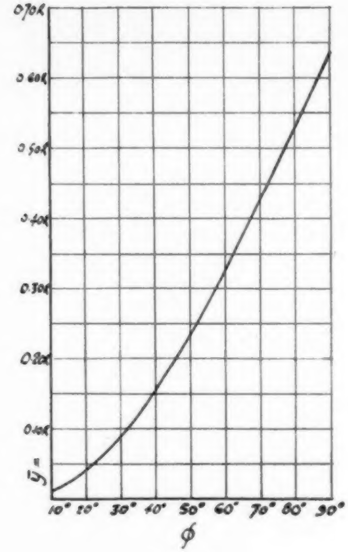


Fig. 4.

$$m_v = \frac{\bar{x}}{I_{yy}} = \frac{EI}{R^2} \cdot \frac{\sin\phi}{\phi - \sin\phi\cos\phi} = v.$$

$$v_v = \frac{1}{I_{yy}} = \frac{EI}{R^3} \cdot \frac{1}{\phi - \sin\phi\cos\phi}.$$

Fig. 3 shows curves for $m_H \dfrac{R^2}{EI}$ and $m_v \dfrac{R^2}{EI}$ for values of ϕ between 10 deg. and 90 deg. Corresponding values of h_H and v_v (and consequently of I_{xx} and I_{yy}) are obtained by dividing by $\bar{y}$ (as given in *Fig.* 4) and $R \sin \phi$ respectively.

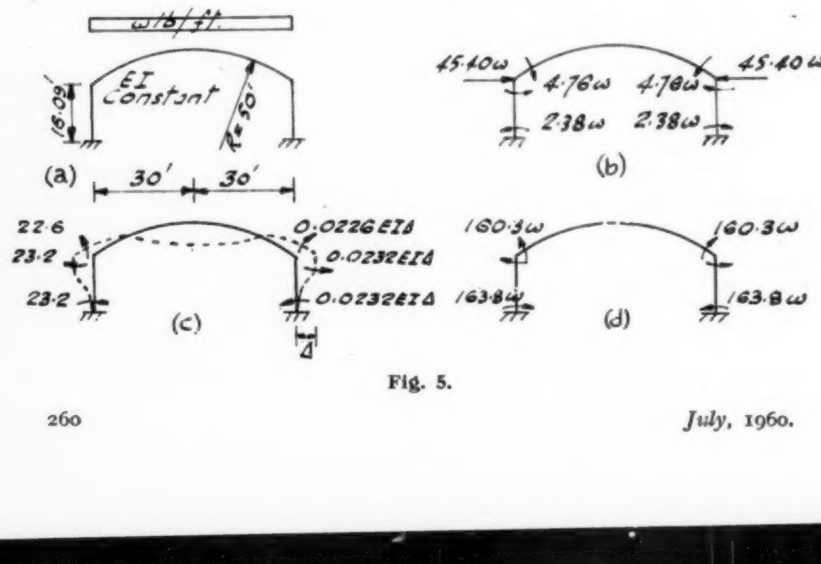


Fig. 5.

EXAMPLE.—The frame in Example 1 in the article by Mr. Chronowicz is acted upon by a distributed load of 2000 lb. per foot on the curved member (*Fig.* 5a).

Sin $\phi = \frac{30}{50}$; therefore $\phi = 0{\cdot}644$ radians $= 36° \ 52'$. Then, if m_s is the statically-determinate bending moment at any section of the column,

$$\int \frac{m_s y ds}{EI} = \frac{150{,}000w}{EI} \text{ and } \int \frac{m_s ds}{EI} = \frac{18750w}{EI}.$$

Therefore $\dfrac{\int \frac{m_s y ds}{EI}}{\int \frac{m_s ds}{EI}} = 8$ ft. From *Fig.* 4, $\bar{y} = 0{\cdot}132R = 6{\cdot}6$ ft., and, from *Fig.* 3, $m_H \frac{R^2}{EI} = 28{\cdot}2$; also $I_{xx} = \frac{578}{EI}$, that is

$$m_{ia} = \frac{\frac{18750w}{EI}}{2 \times \frac{50\phi}{EI}} - \frac{\frac{18750w}{EI} \times 1{\cdot}4 \times 6{\cdot}6}{\frac{578}{EI}} = -\ 8{\cdot}2w.$$

Therefore the fixed-end moment due to the loading is $M_{AB} = m_{SA} - m_{iA} = +\ 8{\cdot}2\,w$ ft.-lb. The horizontal thrust at A due to the uniformly-distributed load is

$$H_{AB} = \frac{M_{xx}}{I_{xx}} = \frac{\frac{18750w}{EI} \times 1{\cdot}4}{\frac{578}{EI}} = 45{\cdot}39w \text{ acting inwardly.}$$

From *Fig.* 2, the moment required to produce unit rotation at A with B fixed is $m = \frac{6{\cdot}75EI}{R} = 0{\cdot}135EI$, and the moment at B is $cm = -\ \frac{2{\cdot}35EI}{R}$, whence the carry-over factor from A to B $= \frac{-\ 2{\cdot}35}{6{\cdot}75} = -\ 0{\cdot}35$. Therefore the distribution factors at A are 0·65 and 0·35, and these are used in the ordinary method of moment distribution of non-sway moments to produce a final moment of $\pm\ 2{\cdot}38w$ at the base of the columns and $\pm\ 4{\cdot}76w$ at A and B as is shown in *Fig.* 5(b). From *Fig.* 3

$$m_H = h = \frac{28{\cdot}2EI}{R^2} = 0{\cdot}0113EI$$

and $\frac{h}{m} = 0{\cdot}084$. The horizontal thrust at A due to the application of balancing moments $(2{\cdot}87w - 0{\cdot}35w + 0{\cdot}04w)$ is $0{\cdot}084 \times 2{\cdot}56w \times 2$, or $0{\cdot}430w$ acting outwardly.

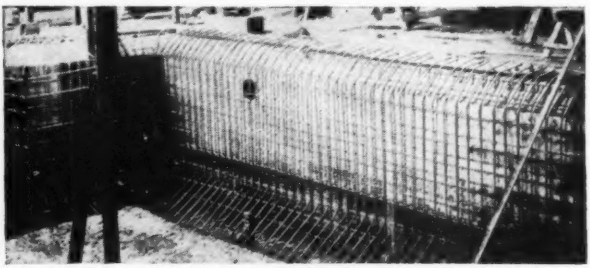
The horizontal force at A (or B) due to the moments from the post is $\frac{7{\cdot}14w}{16{\cdot}09} = 0{\cdot}444w$ acting inwardly. Thus the propping forces at A and B are $45{\cdot}40w$ acting inwardly.

Applying equal and opposite arbitrary sway forces, displacements of Δ outwardly at A and B are produced. Moments of $\frac{6EI\Delta}{16{\cdot}09^2} = 0{\cdot}0232EI\Delta$ will occur at each end of the columns (*Fig.* 5c). Since m_H is produced by unit displacement of one end of the curved member, a displacement of Δ at each end of the member will produce moments of $2m_H\Delta = 0{\cdot}0226EI\Delta$ at each end of the rib. Distributing these moments in the usual way and applying a factor to correct for the actual sway forces, the final sway moments are produced and are $\pm$ 166·2w and $\pm$ 165·05w respectively. Thus the final moments are $\pm$ 163·8w and $\pm$ 160·3w as in *Fig.* 5(d), that is $\pm$ 327,600 ft.-lb. at the bases and $\pm$ 320,600 ft.-lb. at the tops of the columns.

FIFTY YEARS AGO.

From "Concrete and Constructional Engineering", July, 1910.

Swimming Pool at the Royal Automobile Club, London.



" This bath is situated in the basement and is 86 ft. 9 in. by 30 ft. 8 in. The depth varies from 4 ft. 4 in. to 8 ft. 10 in. A subway is constructed on all four sides immediately on the outside of the bath; the space that could be utilised for this purpose was limited, and consequently it was necessary to keep the wall of the bath as thin as possible. Had any material other than reinforced concrete been adopted the walls would necessarily have been so thick that the space available for the subway would have been reduced to such an extent as to render it useless. The space for the subway and bath was firstly prepared, and the asphalt damp course, which passes under the entire building, was completed, and then the concrete work was commenced. The thickness of the reinforced work is 18 in. for the bottom and 12 in. for the walls. The reinforcement to the bottom consisted of ⅝-in. square indented steel bars spaced at 6-in. centres across the width of the bath, with similar bars placed longitudinally at 12-in. centres. These bars were placed in both the top and bottom surface of the concrete. The walls were reinforced with vertical bars ½ in. square spaced at 6-in. centres, with similar bars placed horizontally at 9-in. centres, these being also put in both the inner and outer surfaces of the concrete. The total weight of the reinforcement was about 40 tons. The concrete was composed of 1 part Portland cement, 2 parts sand, and 4 parts of ballast. A layer of asphalt was applied over the inner surface of the bath, and the bath when finished will be lined with white Sicilian marble 1½ in. thick laid in slabs 3 ft. wide and the full depth of the bath without horizontal joints." [The pool is still in use, and there are no leaks despite a crack half-way along the length of the pool. The linings are perfect. Maintenance has been negligible.—Ed.]

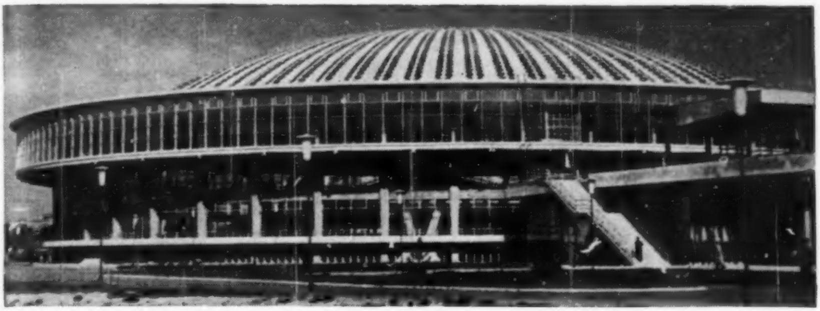


FIG. 1.

A Large Dome at Belgrade.

By B. ŽEŽELJ.

THE main exhibition hall of the Belgrade Fair (*Fig.* 1) was the largest circular hall at the exhibition and is now used for sporting events and public meetings. The original design, which was described in this journal for January, 1956, was subsequently modified and the building is constructed as described in the following.

The hall covers an area of nearly 100,000 sq. ft. and is 107 m. (351 ft.) in diameter and 27·8 m. (91 ft.) high. Two galleries around the perimeter are 4·7 m. (15 ft. 4¾ in.) and 8·7 m. (28 ft. 6 in.) above ground and are enclosed by glass walls on the outside. An ellipsoidal area in the middle of the ground floor is lower than the surrounding area under which are changing rooms and store rooms. The hall is in three parts which are structurally separate, namely the dome and its supports, the galleries and their supports, and the basement rooms and ground floor.

The Dome.

The dome has a central circular cap supported on eighty semi-arches of I-section which bear on a circumferential hollow beam 97·4 m. (319·5 ft.) in diameter. Three intermediate beams between the crown and the main circumferential beam stiffen the arches laterally. The spaces between the semi-arches are filled with curved slabs of lightweight concrete which contain circular roof-lights (*Fig.* 3). The circumferential beam is supported by eight pairs of inclined columns, each pair forming a V. The cap at the crown of the dome comprises two slabs 8 cm. (3·14 in.) thick and 80 cm. (2 ft. 7 in.) apart which are stiffened by three circular and eight radial beams between the slabs (*Fig.* 2). The tangent to the surface at the edge of the cap is parallel to the line of thrust of the arches. This part of the dome is reinforced to resist the unbalanced forces which occur during the erection of the dome. Under the greatest symmetrical load assumed in the design the stress in the concrete is 81 kg. per square centimetre (1152 lb. per square inch) at the upper edge of the crown and 3 kg. per square centimetre (42·67 lb. per square inch) at the lower edge. Under symmetrical load the stress in the semi-arches varies from 22 kg. to 30 kg. per square centimetre (313 lb. to 427 lb. per square inch) in compression, and these members are designed to resist the effects of unsymmetrical loading.

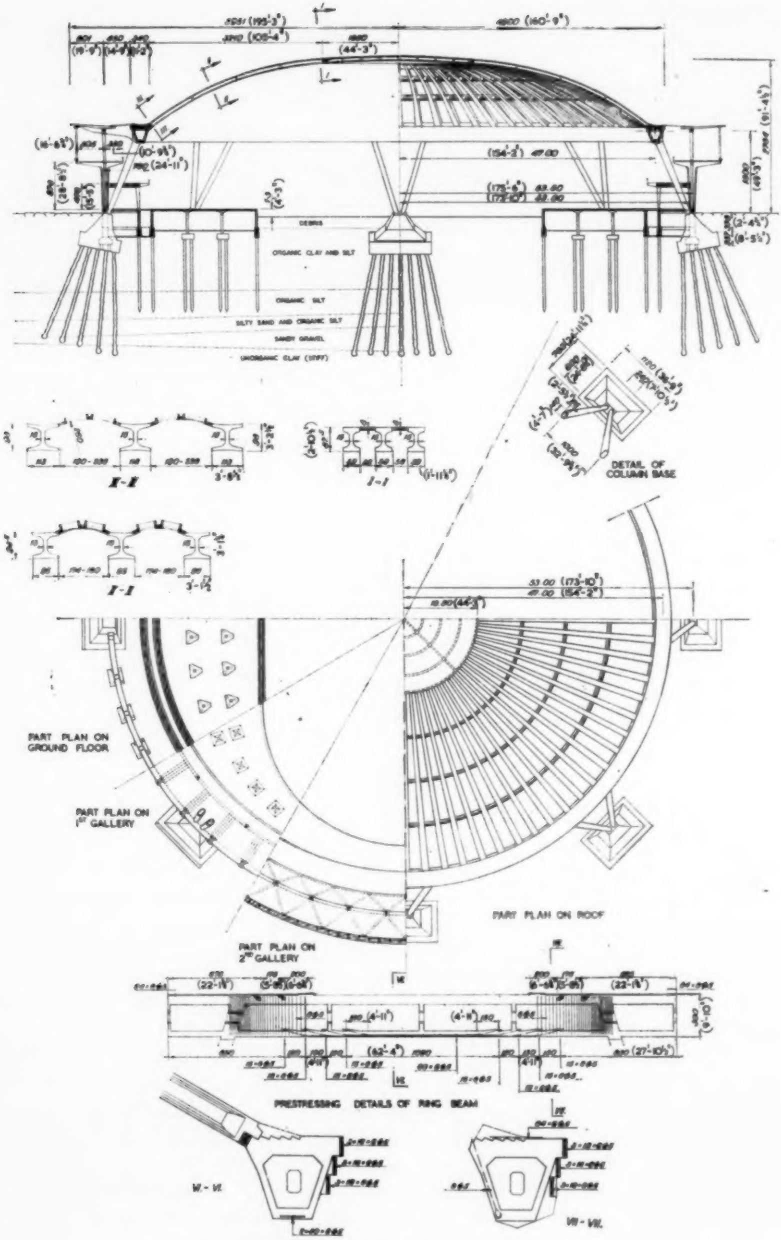


Fig. 2.—Plans, Sections, and Details.

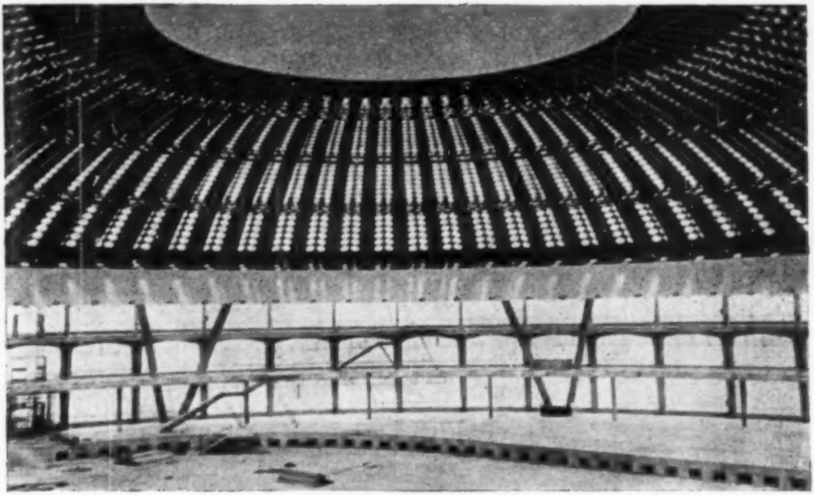


Fig. 3.—The Interior.

The structure is statically indeterminate to a high degree. Most of the load on the dome is permanent and symmetrical about the centre. In the design it was assumed that the circumferential beam would not deflect vertically, that the semi-arches were fixed at the lower ends and hinged at the upper ends, and that the intermediate horizontal ties between the arches were hinged. The magnitudes of the stresses were then calculated for these conditions assuming that the symmetrical load would be the weight of the dome (about 75 per cent. of the total) and of snow uniformly distributed on the dome. A model of one-tenth of the full size (*Fig.* 4) was used to aid the estimation of the effects of unsymmetrical load and wind. The model was of prestressed concrete with elastic properties as similar to those of the dome as was practicable. The deflections produced by symmetrical and unsymmetrical loads and horizontal forces were measured, and by analogy the corresponding deflections of the structure were estimated. The

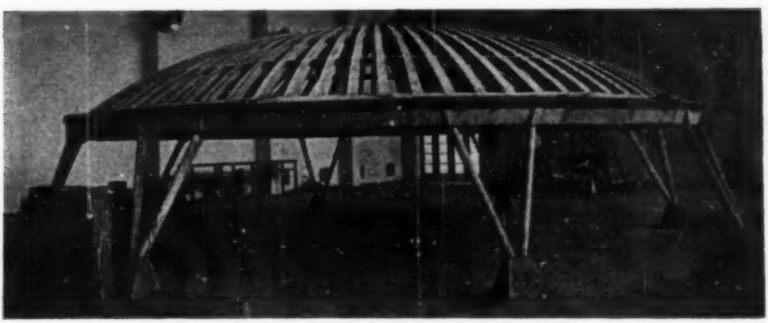


Fig. 4.—The Model.

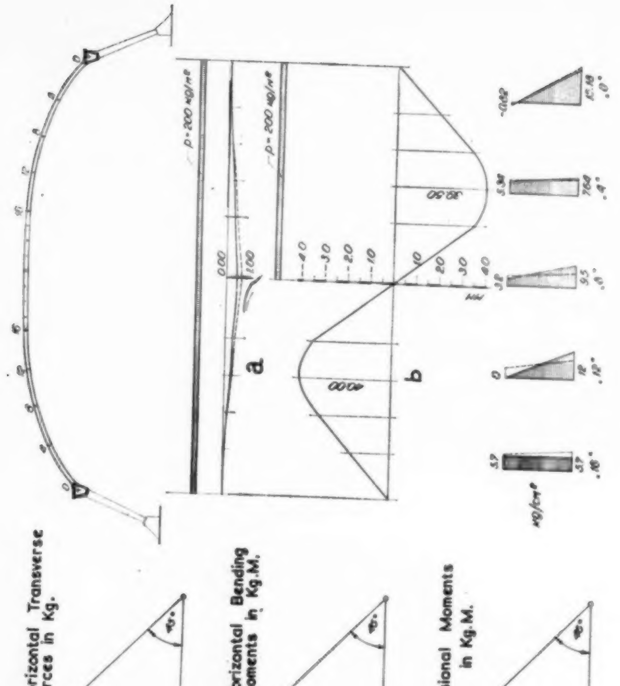


Fig. 6.—Deflection of Model.

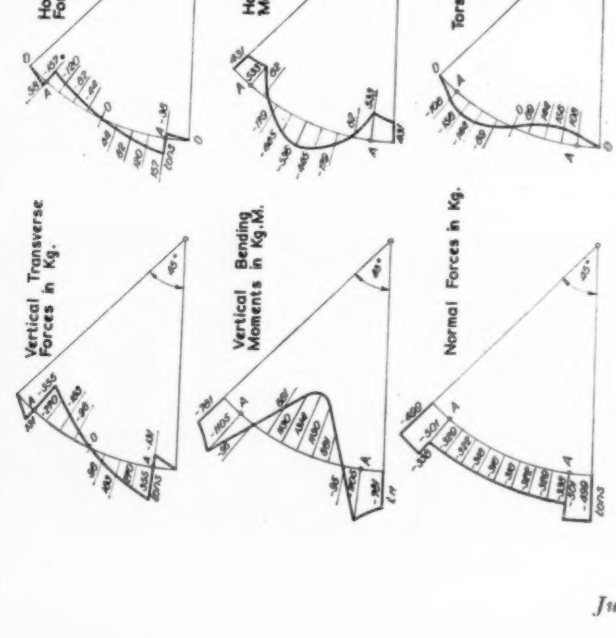


Fig. 5.—Influence Lines.

weight of the model dome was responsible for only 7 per cent. of the total stress while the weight of the large dome produced 70 per cent. of the stress. Therefore, since the model was so much lighter than the structure when compared with the relative strengths of the members, an additional load of 200 kg. per square metre (41 lb. per square foot) was applied to simulate the true conditions. Some of the results of the tests are shown in *Fig.* 6. At (a) the deflections due to the weight of the dome only are shown for ten positions along a diameter. At (b) are shown the deflections when only one-half of the dome is loaded and for this case the stresses measured at the same points are also given.

From the results of the tests on the model it was concluded that when the loads were symmetrical the compressive stresses would be the critical factor. Under unsymmetrical loading, however, bending moments and tensile stress were produced. When the greatest unsymmetrical load was applied (2·66 times the load due to snow) the tensile stresses in the model arches were not large. In the structure itself they are even smaller, and the factor of safety is not exceeded.

The Circumferential Beam.

The circumferential beam is hollow (*Fig.* 2) and trapezoidal in shape, so as to provide resistance to horizontal and vertical forces. The influence lines in the vertical and horizontal directions are shown in *Fig.* 5. Stresses up to 105 kg. per square centimetre (1494 lb. per square inch) occur at the edges, and vertical prestressing is necessary in some parts to reduce the tensile stresses.

The beam is prestressed in three ways, as follows. Circumferential prestressing to resist horizontal forces is applied by 142 cables, each comprising six wires of 5 mm. (0·2 in.) diameter, on the outside of the beam and spaced symmetrically about the centroid of the section (*Fig.* 2). The total residual tension in the cables is 1420 tons. The cables are kept in place by steel spacers projecting from the concrete, and each cable was tensioned simultaneously at six points on the circumference. After they were tensioned and anchored, these cables were covered with vibrated concrete. To resist vertical bending moments, sixty-four cables were tensioned over the supports and sixty cables in the lower part of the beam between the supports. The cables over the supports apply a prestressing force of 640 tons to resist the negative bending moments; the cables are on the top of the beam and are kept in place around the curve by steel spacers embedded in the concrete. At one end the cables are embedded in the beam and at the other end they are tensioned and anchored in precast anchorages embedded in the beam. In the bottom of the beam the cables between the supports pass through ducts formed by inflatable rubber tubes and exert a prestressing force of 600 tons. The third series of cables was applied vertically in the inner wall of the beam to resist oblique tensile forces near the supports.

Columns and Foundations.

The inclination of the columns towards the centre permitted a reduction in the size of the dome and provided lateral stability. By inclining the columns in each pair away from each other the spans of the circumferential beam are reduced and the resistance of the structure to twisting about the vertical axis is increased. Each column is an inverted and truncated ellipsoidal cone. The greatest stress in the concrete is 90 kg. per square centimetre (1280 lb. per square inch).

The columns are supported in pairs on eight bases each of which is cast around the tops of thirty-five bored piles 40·5 cm. (16 in.) in diameter which bear on clay 20 m. to 23 m. (66 ft. to 98 ft.) below the surface of the ground. The dead load on each pile is 55 tons and the greatest load due to the effect of unbalanced loads is 65 tons. Although these piles are sloped to resist horizontal components of forces, the bases are also connected by a prestressed peripheral beam 107 m. (351 ft.) in diameter and at a depth of 2 m. (6 ft. 6 in.). The beam is prestressed with thirty cables each comprising six 5-mm. (0·2-in.) wires which exert a total force of 300 tons. The wires are painted with bitumen and protected by a coating of gunite. Also connected by this beam are thirty-two bases supporting the columns of the gallery. Each base is supported on four perpendicular bored piles. The foundations (*Fig.* 2) for the store-rooms, walls, and corridors are supported on precast prestressed piles which are driven into the layer of organic silt.

The Galleries.

The galleries are divided by expansion joints into parts about 60 m. (196·8 ft.) long. The space between the dome and the galleries is covered by lightweight slabs so that the dome and galleries can deflect independently.

The two galleries and their roof are supported on columns and are independent of the dome. The columns which are 7 m. (23 ft.) apart and have a cruciform

Fig. 7.—The Galleries.

cross section are connected around the building by reinforcement in the slab of the lower gallery, the splice-bars for which can be seen in *Fig.* 7. The columns are stabilised in a radial direction by beams at this level and by columns at the inner edge of the gallery (*Fig.* 2). The upper gallery is supported by cantilevers projecting radially from the columns (*Fig.* 7). Above the upper gallery the columns are of rectangular cross section and support the roof. The columns are of prestressed concrete cast in place. The lower gallery and the roof over the upper

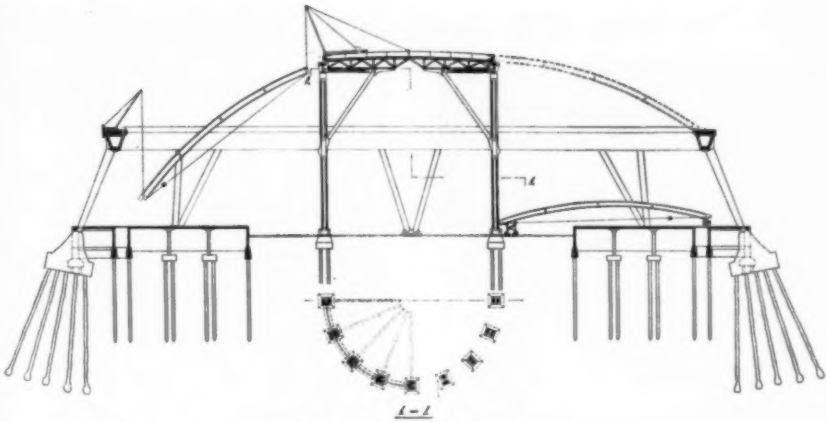


Fig. 8.—Method of Erection.

gallery were precast and prestressed and the floor over the basement is of flat-slab construction. The total cost of the dome and its supports and foundations was about equal to the cost of the remainder of the building.

Materials.

The concrete of the dome, the circumferential beam, and the columns had a crushing strength at twenty-eight days of at least 450 kg. per square centimetre (6400 lb. per square inch) and contained at least 350 kg. of cement per cubic metre of concrete (590 lb. per cubic yard). The other structural concrete had a strength at twenty-eight days of at least 300 kg. per square centimetre (4267 lb. per square inch). The prestressing wire has a breaking strength of 150 kg. per square millimetre (95·25 tons per square inch) and a yield strength of 125 kg. per square millimetre (79·37 tons per square inch). The initial stress was 105 kg. per square millimetre (66·67 tons per square inch) which resulted in a residual stress of 85 kg. per square millimetre (54 tons per square inch). The prestressing was done by a method developed by the writer.

Construction.

The columns and the circumferential beam were cast in place and, because the soil near the surface is of poor quality, the shuttering was supported on scaffolding resting on slabs on the ground. The beam was cast in stages—first the lower slab, then the walls, and then the top slab, parts of which were left open near the columns and were closed later. The beam was prestressed in stages: first the middle group of circumferential cables was tensioned, then the cables in the bottom, then the cables at the top, and finally the remaining two groups of circumferential cables.

It was decided that the poor quality of the ground and the expense of large quantities of shuttering precluded the casting of the dome in place and that precast members should be used. A scaffolding of precast prestressed concrete supported on piles was erected in the middle of the building (*Fig.* 8) and covered

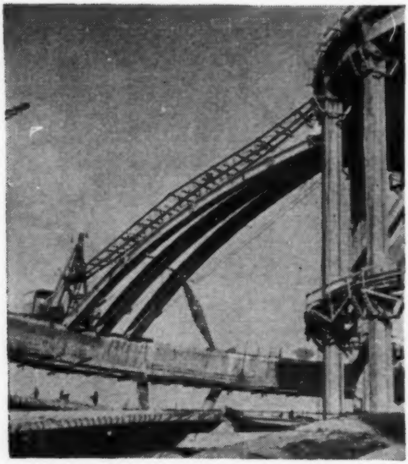


Fig. 9.—The Equipment for Erecting the Ribs.

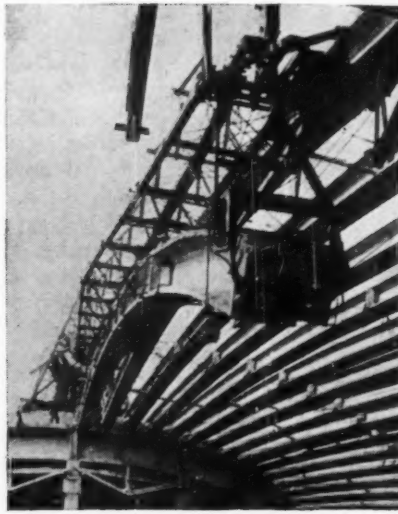


Fig. 10.—The Stirrups for Erecting the Ribs.

only 6·4 per cent. of the area covered by the dome. On this scaffolding the central cap of the dome was cast and a crane was supported above the cap to lift one end of the semi-arches.

The semi-arches were precast on the ground directly under their position in the dome. A tie comprising four cables each of six wires 5 mm. (0·2 in.) in diameter was tensioned between the ends of each semi-arch to resist the horizontal components of the weight. These members were lifted into position by means of two cranes (*Fig.* 9). To prevent them from turning over while they were suspended at the ends only, a steel lattice (*Fig.* 10) was fixed between the stirrups by which the member was suspended and was connected to the arch by ties. Three semi-arches were erected daily by this means. When all the arches were in place and the joints concreted, the tension in the ties was released simultaneously by forty men who allowed sand to escape gradually from pots incorporated in the cables. The supports were then removed from the central cap and a deflection of 27 mm. (1·063 in.) was noted which agreed well with the calculated value.

The architect is M. Pantović. The structure was designed by the writer, and the calculations were made by M. B. Petrović and M. D. Ćertić. The contractors were Trudbenik.

White and Coloured Aggregates.

A BOOKLET* entitled "Sources of White and Coloured Aggregates in Great Britain (1959)" was published recently by the Road Research Laboratory of the Department of Scientific and Industrial Research. The sources of the best naturally coloured roadstones in production in 1959 and which are in the collection at the Laboratory are given, the stones being classified broadly as blue, green, red, yellow, dark grey, light (best) and light (poor). Other lists give the suppliers of white aggregates, and approximate prices, under and over ⅛ in. in size.

* Road Note No. 25. (London: H.M.S.O. 1960. Price 1*s*. 3*d*.)

Design of Slabs by the Yield-line Method.—II.*

By A. J. ASHDOWN, A.M.I.Struct.E.

Rectangular Slabs Supported on Three Sides.

If a slab carrying a uniformly-distributed load is unsupported along one of the shorter sides, the pattern of cracks is likely to be as shown in *Fig.* 6 in which a is greater than b. If the slab at the longitudinal crack is assumed to be deflected an amount δ, the work done is as follows.

Virtual external work: $U = \frac{1}{3}wbx\delta + \frac{1}{2}wb(a - x)\delta = \frac{1}{6}wb(3a - x)\delta.$

Virtual internal work:
$$\frac{V}{\delta} = \frac{(m_3 + m_4)b}{x} + \frac{(m_1 + m)x}{y} + \frac{(m_2 + m)x}{b - y}$$
$$+ \frac{(m_2 + m)(a - x)}{b - y} + \frac{(m_1 + m)(a - x)}{y} \quad \text{(b)}$$

Let $m_1 = i_1 m$, $m_2 = i_2 m$, $m_3 = i_3 m$ and $m_4 = i_4 m$. An expression for y

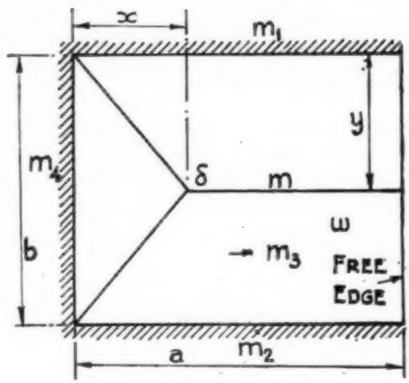


Fig. 6.

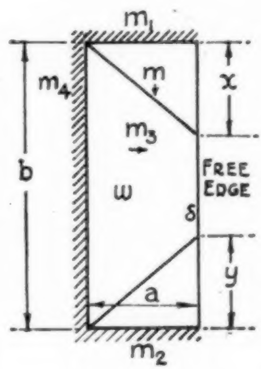


Fig. 7.

can be derived by differentiating the last two terms in (b) and equating the differential to zero, whence

$$y = b\frac{\sqrt{i_1 + 1}}{\sqrt{i_1 + 1} + \sqrt{i_2 + 1}} \quad . \quad . \quad . \quad . \quad (12)$$

The diagonal cracks are not influenced by the work in $(a - x)$, so that the last two terms in (b) are omitted when differentiating to determine the value of x. Since $V = m\delta\left[\frac{(i_3 + i_4)b}{x} + \frac{(i_1 + 1)x}{y} + \frac{(i_2 + 1)x}{b - y}\right]$, from $\frac{dV}{dy} = 0$,

$$x = \sqrt{\frac{by(b - y)(i_3 + i_4)}{(b - y)(i_1 + 1) + y(i_2 + 1)}} \quad . \quad . \quad . \quad (13)$$

*Continued from June, 1960.

If $m_1 = m_2$, $y = \frac{1}{2}b$ and $x = \frac{b}{2}\sqrt{\frac{i_3 + i_4}{1 + i_2}}$. . . . . . . (14)

Equating U and V and reducing,

$$wb(3a - x) = 6m\left[\frac{(i_3 + i_4)b}{x} + \frac{(i_1 + 1)a}{y} + \frac{(i_3 + 1)a}{b - y}\right]. \quad . \quad (15)$$

To determine the greatest load the slab can support substitute the values of x and y given by (12) and (14).

If $m_1 = m_2$, $y = \frac{1}{2}b$, then (15) can be reduced to

$$wxb^2(3a - x) = 6m[(i_3 + i_4)b^2 + (i_2 + 1)4ax]. \quad . \quad . \quad (15a)$$

Generally at the free end

$$m = \frac{wby(b - y)}{2[(b - y)(i_1 + 1) + y(i_2 + 1)]}. \quad . \quad . \quad . \quad (15b)$$

If $m_1 = m_2$, then $\quad m = \frac{wb^2}{8(i_2 + 1)}$. . . . . (15c)

If a is less than $\frac{1}{2}b$ as in *Fig.* 7 the analysis proceeds as follows.

Virtual external work:

$$U = \frac{wxa\delta}{3} + \frac{wya\delta}{3} + w(b - x - y)\frac{a\delta}{2} = \delta wa\left(\frac{b}{2} - \frac{x + y}{6}\right).$$

Virtual internal work:

$$V = \left[(m_4 + m_3)\frac{b}{a} + (m_1 + m)\frac{a}{x} + (m_2 + m)\frac{a}{y}\right]\delta.$$

If $m_1 = i_1 m$, etc. as before,

$$V = \delta m\left[(i_4 + i_3)\frac{b}{a} + (i_1 + 1)\frac{a}{x} + (i_2 + 1)\frac{a}{y}\right].$$

Therefore $\quad wa(3b - x - y) = 6m\left[(i_4 + i_3)\frac{b}{a} + (i_1 + 1)\frac{a}{x} + (i_2 + 1)\frac{a}{y}\right].$

Using the previous results,

$$x = \frac{a}{2}\sqrt{\frac{i_1 + 1}{i_3 + i_4}} \quad \text{and} \quad y = \frac{a}{2}\sqrt{\frac{i_2 + 1}{i_3 + i_4}}. \quad . \quad . \quad . \quad (16)$$

If a is long compared with b (*Fig.* 8), there is a limit to the application of formulæ (13) and (15), since the deflection tends to increase towards the free end, and is limited by the capacity of the reinforcement to stretch beyond the yield point and the resistance to hogging cracks along diagonal lines. The virtual internal work within zb is given by

$$V = m\left[4(i_1 + i_5)\frac{z\delta_1}{b} + 4(1 + i_5)\frac{z}{b}(\delta_2 - 2\delta_1) + (i_3 + i_5)\frac{b}{z}(\delta_2 - \delta_1)\right].$$

When $\frac{\delta V}{\delta z} = 0$, $4(i_1 + i_5)\delta_1 + 4(1 + i_5)(\delta_2 - 2\delta_1) - (i_3 + i_5)\frac{b^2}{z^2}(\delta_2 - \delta_1) = 0$

whence $\quad z = \frac{b}{2}\sqrt{\frac{(i_3 + i_5)(\delta_2 - \delta_1)}{(i_1 + i_5)\delta_1 + (1 + i_5)(\delta_2 - 2\delta_1)}}$. . . (17)

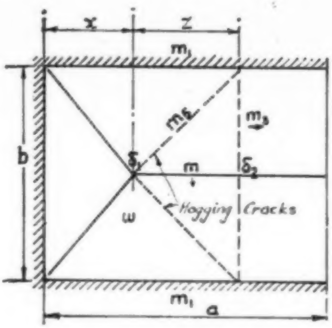


Fig. 8.

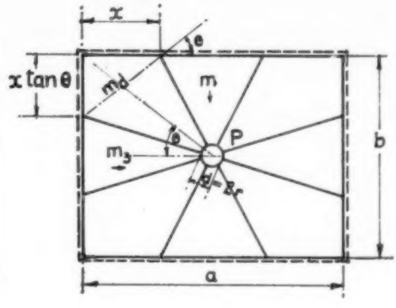


Fig. 9.

If $\delta_2 = 2\delta_1$, z varies between $0{\cdot}5b$ and $0{\cdot}55b$; if $m_1 = 0$, z varies between $0{\cdot}7b$ and $0{\cdot}78b$.

If $\delta_2 = 1{\cdot}5\delta_1$, z varies between $0{\cdot}5b$ and $0{\cdot}55b$. If $w_1 = 0$, z will be infinite and hogging does not occur.

Slabs Freely Supported along Four Edges and Subjected to a Central Concentrated Load.

Tests show that if a rectangular slab is freely supported along four edges (*Fig.* 9) and is subjected to a central concentrated load the cracks do not radiate to the corners, but extend to points some distance on either side of the corners; this action is due, no doubt, to the torsional stiffness at the corners.

Let $m_3 = im$, and $m_d = m\cos^2\theta + m_3\sin^2\theta = m(\cos^2\theta + i\sin^2\theta)$. Since $\tan\theta = \dfrac{b}{a}$, $\cos^2\theta = \dfrac{a^2}{a^2+b^2}$, $\sin^2\theta = \dfrac{b^2}{a^2+b^2}$ and $\dfrac{4m_d x\sec\theta}{\dfrac{a}{2\cos\theta} - \dfrac{x\sec\theta}{2}} = \dfrac{8m_d x}{a-x}$, the equation of work, if the size of the loaded area is omitted, is

$$P = 4m\frac{(a-2x)}{b} + 4im\frac{(ab-2bx)}{a^2} + \frac{8m_d x}{a-x}. \qquad (18)$$

For a minimum value of x,

$$\frac{dP}{dx} = 0 = -\frac{8m}{b} - \frac{8imb}{a^2} + \frac{8m_d a}{(a-x)^2}.$$

Substituting for m_d and reducing,

$$\frac{a^2+ib^2}{a^2b} = \frac{a(a^2+ib^2)}{(a^2+b^2)(a-x)^2};\ \text{hence } a^3b = (a^2+b^2)(a-x)^2,$$

and $$a - x = a\sqrt{\frac{ab}{(a^2+b^2)}}.$$

Therefore $$x = a\left(1 - \sqrt{\frac{ab}{a^2+b^2}}\right). \qquad (19)$$

which owing to the assumed pattern of cracks is independent of m_3/m. If $b = a$,

$$x = a\left(1 - \frac{\sqrt{2}}{2}\right). \qquad (20)$$

For the general case, allowing for the size of the loaded area

$$P = 4m\left[\frac{a - 2x}{b} + \frac{i(ab - 2bx)}{a^2} + \frac{2x(a^2 + ib^2)}{(a - x)(a^2 + b^2)}\right]\frac{b}{b - d}. \qquad (21)$$

If the slab is square $\quad P = \dfrac{8m(1 + i)(\sqrt{2} - 1)a}{a - d}. \qquad (22)$

If the slab is square and isotropic, $P = \dfrac{6{\cdot}63ma}{a - d}. \qquad (22a)$

Slabs Freely Supported along Three Edges and Subjected to a Central Concentrated Load.

Consider the slab shown in *Fig.* 10.

Let $m_3 = im$, $m_d = m(\cos^2\theta + i\sin^2\theta)$; $\tan\theta = \dfrac{b}{2c}$, $\cos^2\theta = \dfrac{c^2}{\frac{1}{4}b^2 + c^2}$ and $\sin^2\theta = \dfrac{\frac{1}{4}b^2}{\frac{1}{4}b^2 + c^2}$.

$$P = \frac{4m(a - x)}{b} + im\frac{(b - 2x\tan\theta)}{c} + \frac{2x\sec\theta m_d}{c\sec\theta - \frac{x}{2}\sec\theta}.$$

Since $\dfrac{2x\sec\theta m_d}{c\sec\theta - \frac{1}{2}x\sec\theta} = \dfrac{4xm_d}{2c - x}$, for the minimum value of P

$$\frac{dP}{dx} = 0 = -\frac{4m}{b} - \frac{2\tan\theta im}{c} + \frac{8cm_d}{(2c - x)^2}.$$

Substituting for m_d

$$\frac{4c^2 + ib^2}{bc^2} = \frac{2c(4c^2 + ib^2)}{(\frac{1}{4}b^2 + c^2)(2c - x)^2}.$$

Therefore $2bc^3 = (\frac{1}{4}b^2 + c^2)(2c - x)^2$, and $2c - x = c\sqrt{\dfrac{2bc}{\frac{1}{4}b^2 + c^2}}$, from which,

$$x = c\left(2 - \sqrt{\frac{8bc}{b^2 + 4c^2}}\right). \qquad (23)$$

Therefore x is independent of $\dfrac{m_3}{m}$. Allowing for the size of the loaded area and reducing,

$$P = m\left[\frac{4(a - x)}{b} + \frac{ib(c - x)}{c^2} + \frac{4x(4c^2 + ib^2)}{(2c - x)(b^2 + 4c^2)}\right]\frac{b}{b + d}. \qquad (24)$$

If a is long compared with b or c a hogging crack is likely to form near the face edge, thereby limiting the effective width of the slab supporting P. The limiting

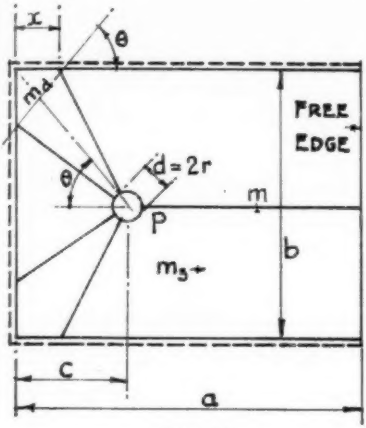


Fig. 10.

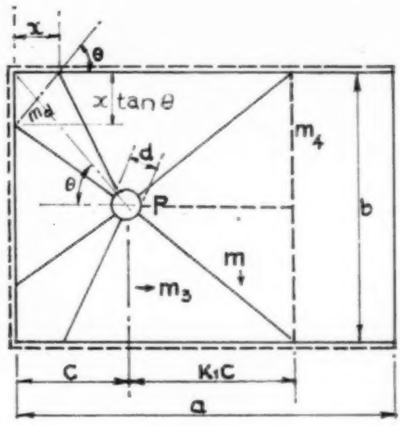


Fig. 11.

width $c + K_1c$ in *Fig.* 11 depends on the resistance to hogging cracking of the longer span of the slab relative to the resistance to sagging cracking of the shorter span. It is shown by formula (23) that the distance x is dependent on c and b only, and is known. Let the cracking moments of resistance $m_3 = i_3m$, $m_4 = i_4m$ and $m_d = m(\cos^2\theta + i_3\sin^2\theta)$, then

$$P = 4[c(1 + K_1) - x]\frac{m}{b} + (b - 2x\tan\theta)i_3\frac{m}{c} + \frac{4xm_d}{2c - x} + b(1 + i_4)\frac{m}{K_1c}.$$

From $\dfrac{dP}{dK_1} = \dfrac{4mc}{b} - m(1 + i_4)\dfrac{b}{cK_1^2} = 0,$

$$K_1 = \frac{b}{2c}\sqrt{1 + i_4} \quad . \quad . \quad . \quad . \quad . \quad (25)$$

and $$P = m\left[\{c(1 - K_1) - x\}\frac{4}{b} + \frac{i_3b(c - x)}{c^2} + \frac{4x(4c^2 + i_3b^2)}{(2c - x)(b^2 + 4c^2)}\right]\frac{b}{b - d}. \quad (26)$$

Slabs Supported along and Continuous over Two Adjacent Sides: Uniformly-distributed Load.

Consider the slab in *Fig.* 12.

The total virtual external work is given by

$$U = \frac{\delta pbx}{3} + \frac{\delta p(a - x)b}{2} = \frac{\delta pb(3a - x)}{6}.$$

The total virtual internal work is given by

$$V = \delta\left[\frac{m_2b}{x} + \frac{m_3b}{x} + \frac{mx}{b} + \frac{m_1x}{b} + \frac{m_1(a - x)}{b}\right].$$

The last term refers to the part of slab beyond the crack and may be neglected. Let $m_1 = i_1m$, $m_2 = i_2m$, $m_3 = i_3m$, $m_4 = i_4m$. Then

$$V = \delta m\left[\frac{b(i_2 + i_3)}{x} + \frac{x(1 + i_1)}{b}\right].$$

For the minimum value of V, $\dfrac{dV}{dx} = 0$, and

$$x = b\sqrt{\frac{i_2 + i_3}{1 + i_1}}. \quad . \quad . \quad . \quad . \quad . \quad . \qquad (27)$$

Equating U to V

$$\frac{pb(3a - x)}{6} = m\left[\frac{b(i_2 + i_3)}{x} + \frac{x + i_1 a}{b}\right].$$

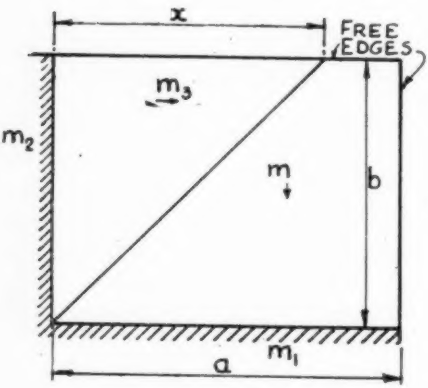


Fig. 12.

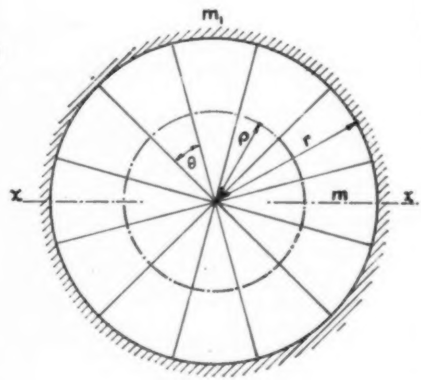


Fig. 13.

Substituting for x gives

$$m = \frac{pb^2}{6}\left[\frac{3a\sqrt{1 + i_1} - b\sqrt{i_2 + i_3}}{ai_1\sqrt{1 + i_1} + b(i_1 + 2)\sqrt{i_2 + i_3}}\right]. \quad . \quad . \quad . \qquad (28)$$

Circular Slabs.

Consider the circular slab in *Fig.* 13 which is supported and fixed around the circumference.

If the slab supports a uniformly distributed load of intensity w covering the circular area of radius ρ

$$2rm_1\int_0^{\pi/2}\cos\theta\,.d\theta + 2mr = 2w\int_0^{\pi/2}\frac{\rho^2\pi r^2}{2\pi r}\cos\theta\,.d\theta - 2w\int_0^{\pi/2}\frac{\rho^2}{2}\,.\,\tfrac{2}{3}\rho\cos\theta\,.d\theta.$$

Then $2m(i_1 + 1) = w\left(\rho^2 - \dfrac{2}{3}\dfrac{\rho^3}{r}\right)$.

Let $P = w\rho^2\pi$; then

$$m = \frac{P\left(1 - \dfrac{2}{3}\dfrac{\rho}{r}\right)}{2\pi(i_1 + 1)}. \quad . \quad . \quad . \quad . \quad . \quad . \qquad (29)$$

If the slab is freely supported $i_1 = 0$ and

$$m = \frac{P}{2\pi}\left(1 - \frac{2}{3}\frac{\rho}{r}\right). \qquad (30)$$

If the entire slab is covered by uniformly distributed load,

$$m = \frac{P}{6\pi(i_1 + 1)}. \qquad (31)$$

If the slab supports a central concentrated load,

$$m = \frac{P}{2\pi(i_1 + 1)}. \qquad (32)$$

If the slab is loaded on a ring of radius ρ,

$$m = \frac{P\left(1 - \frac{\rho}{r}\right)}{2\pi(i_1 + 1)}. \qquad (33)$$

Triangular Slabs.

The bending moment m on a triangular slab (*Fig.* 14) freely supported along three edges and carrying a uniform load of intensity w is $\frac{1}{6}wr^2$, in which r is

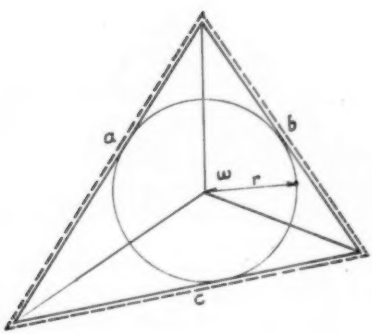


Fig. 14.

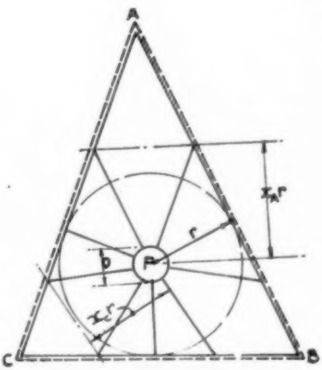


Fig. 15.

the radius of the inscribed circle, or if s is half the sum of the sides a, b and c,

$$m = \frac{w(s-a)(s-b)(s-c)}{6s} \text{ per unit length.} \qquad (34)$$

If the slab is continuous over the edges where the moment of resistance is im, then

$$m = \frac{w(s-a)(s-b)(s-c)}{6(1+i)s} \text{ per unit length.} \qquad (35)$$

In the case of a freely supported triangular slab supporting a concentrated load P at the centre of the inscribed circle, the pattern of cracks is likely to be as shown in *Fig.* 15. The distribution of the reactions is assumed to be

parabolic along each side with the greatest ordinate at the points of contact with the inscribed circle. Then

$$x_A r = r \sin \tfrac{1}{2}A + 0{\cdot}4r \cot \tfrac{1}{2}A . \cos \tfrac{1}{2}A \quad \text{and} \quad P = 2m\left(\frac{\cos \frac{1}{2}A}{x_A} + \frac{\cos \frac{1}{2}B}{x_B} + \frac{\cos \frac{1}{2}C}{x_C}\right)$$

if the size of the loaded area is neglected. Therefore

$$m = \frac{P}{2\left(\frac{\cos \frac{1}{2}A}{x_A} + \frac{\cos \frac{1}{2}B}{x_B} + \frac{\cos \frac{1}{2}C}{x_C}\right)} \text{ per unit length.} \qquad (36)$$

If the size of the loaded area is taken into account, $P = \dfrac{4{\cdot}73m}{1 - \dfrac{D}{2{\cdot}2r}}$ approximately, where D is the diameter of the loaded area. (Note that D is used in *Fig.* 4 of Part I to indicate the distance between two concentrated loads on a slab.)

Therefore

$$m = \frac{P}{4{\cdot}73}\left(1 - \frac{D}{2{\cdot}2r}\right) \text{ per unit length.} \qquad (37)$$

If the slab is continuous over the three edges where the moment of resistance is im

$$m = \frac{P\left(1 - \frac{D}{2{\cdot}2r}\right)}{4{\cdot}73(1 + i)} \text{ per unit length.} \qquad (38)$$

If m is the yield-moment of resistance, formula (38) transposed gives the concentrated load at which the slab will fail in bending. The load causing failure due to shearing may be less than this value of P.

Tests.

The tests made by Professor A. J. Ockleston which were referred to in Part I of this article (pages 221 and 223, June, 1960) are described in a paper by Professor Ockleston entitled "Loading Tests on Reinforced Concrete Slabs Spanning in Two Directions. Tests on the old Dental Hospital, Johannesburg" (Paper No. 6. Johannesburg Portland Cement Institute, October, 1958) and in the Journal of the Institution of Structural Engineers for October, 1955.

These papers deal with full-scale loading tests which were carried out during the course of demolition of a three-story reinforced concrete framed building which had been in service for about ten years. The tests were made on slabs spanning in two directions and forming part of a large beam-and-slab floor. Uniform loading was applied to single panels and to two adjacent panels and were continued until failure occurred. For loads within the working range, the floor behaved elastically, and the deflections of, and the stresses in, the slabs were much less than normal design calculations indicated. The deflections of the supporting beams were relatively large and the general behaviour of the floor differed materially from that normally assumed in design. The mechanisms of failure were in fair agreement with those predicted by the yield-line theory.

A 3,000,000-gallon Reservoir with Precast Prestressed Walls.

A RESERVOIR of 3,000,000 gallons capacity at St. Leonards Hill, Windsor, is circular in plan and has an internal diameter of 160 ft. (*Fig.* 1). The depth of the water may be up to 24 ft. A concentric internal wall 10 ft. high is provided so that water to the extent of a quarter of the total capacity can be stored during inspection and cleaning of the remainder of the reservoir. The roof is a reinforced concrete dome having a rise of 20 ft., and is supported on the outer wall. The outer wall is constructed of precast slabs placed vertically, and is prestressed vertically and circumferentially. The vertical prestressing is by means of $1\frac{1}{8}$-in. steel-alloy bars in metal sheaths embedded in the wall. Circumferential prestressing is by means of cables each of twelve wires of 0·276-in. diameter contained in 2-in. diameter metal sheaths embedded in the wall. The cables are anchored at six buttresses spaced equidistant around the wall, each cable extending through an arc subtending 120 deg. Soil is to be banked against the wall to its full height and the roof will be covered with a layer of soil and gravel 1 ft. thick.

Fig. 1.

Design.

The bottom of the outer wall rests in a rebate in the footing (*Fig.* 2) and, to reduce the bending moment, the wall was allowed to move inwards under the action of initial prestress. Dry mortar and a waterproof seal, which were then placed in the rebate, prevented the foot of the wall from moving farther inwards under the action of the final prestress. When the reservoir is full, there will be a residual compressive stress circumferentially in the wall of about 150 lb. per square inch under the most adverse conditions of loading.

The ring-beam at the top of the wall and forming the springing of the dome was prestressed in stages so that the compressive stresses in the beam were never excessive, although a large final prestressing force is required to resist the heavy load on the roof. In the first stage eighteen wires were tensioned when the wall was prestressed. More wires will be tensioned after the roof is cast. The remaining wires will be tensioned as the roof is covered with soil. The tensile stresses will be limited to 150 lb. per square inch at each stage. The domical roof is in the form of a part of a sphere of 170-ft. radius and is 4 in. thick except for the outer annulus 15 ft. 9 in. wide, in which the thickness increases to 9 in. at the ring-beam. The position of the line of thrust of the dome with respect to the centroid of the ring-beam is such that the radial bending moments due to the edge effects are small. The design allows for the possibility of the soil being removed from the

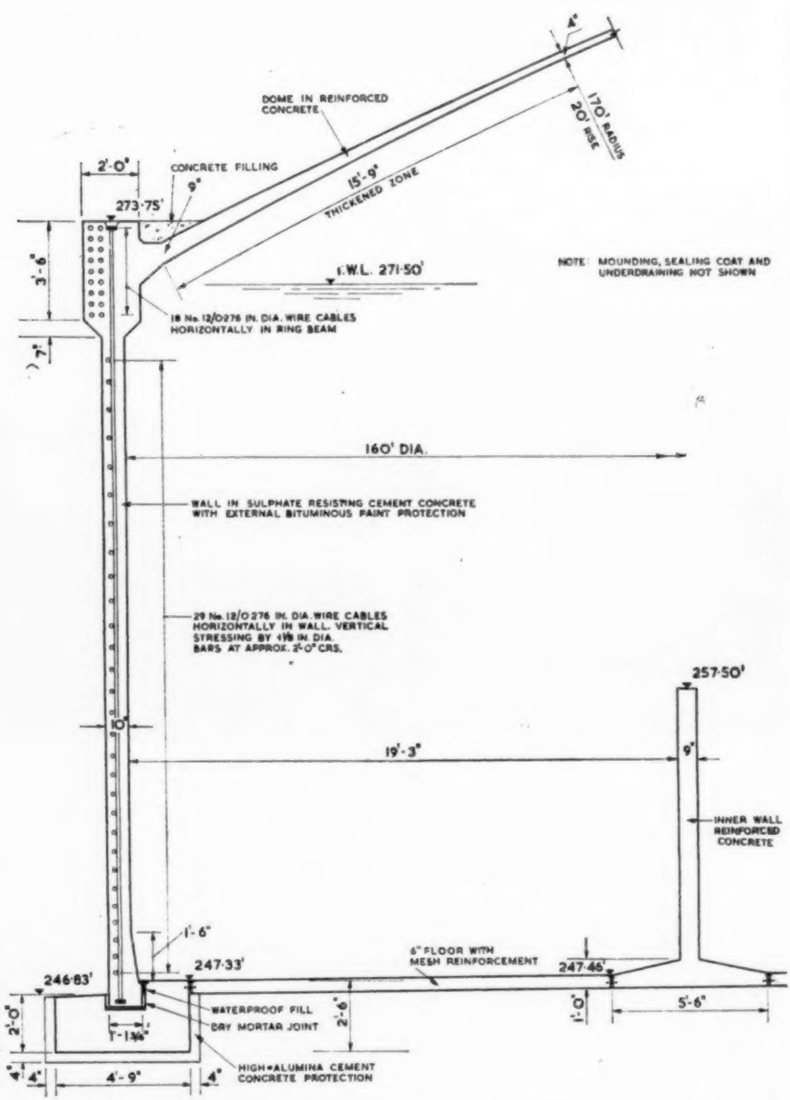


Fig. 2.—Part Section Through Reservoir.

roof. Mild-steel reinforcement is provided in the dome to resist stresses due to shrinking.

Construction.

The ground has a sulphate content of about 3 per cent. Sulphate-resisting cement is used for all concrete except that of the internal wall above the footing.

Fig. 3.—Erecting the Wall.

The exterior of the reservoir is to be coated with bituminous paint. Most of the concrete was delivered to the site ready-mixed. For the outer wall, ring-beam and dome the mixture was 1 : 1·2 : 2·4 and for the footings, floor and inner wall it was 1 : 1½ : 3, the concretes having crushing strengths at 28 days of at least 6500 lb. and 3750 lb. per square inch respectively.

The floor is 6 in. thick and is separated from the footings of the walls. Water-bars are provided at the junctions of the floor and walls. Alternate wall slabs, which include the ring-beam and each of which are about 17 tons in weight, were cast horizontally in moulds on the floor and prestressed by means of steel-alloy bars. The initial tension in each bar was about 33 tons. The slabs were raised into an upright position by means of a gantry built for the purpose (*Fig.* 3), and were propped in position while the remaining slabs were being cast (*Fig.* 4). Three slabs could be cast in a day, but since each mould was used several times the casting of the forty-eight slabs occupied about two months and the erection about six weeks.

There was a gap of about 8 in. between adjacent slabs when erected. When the slabs had been erected the ducts for the horizontal cables were cleared where necessary by passing through the duct a pneumatic tool with rotary cutters (*Fig.* 5) which planed out any irregularities. When all the slabs had been erected and aligned, the vertical prestress was relaxed by slackening the nuts at the top of the bars. The cables were inserted in the circumferential ducts which were then made continuous across the gaps, the gaps were

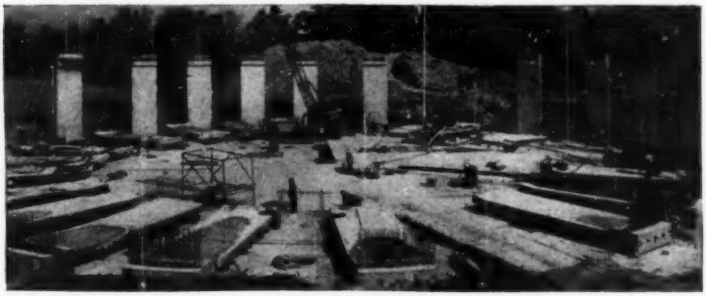


Fig. 4.—Casting and Erecting Wall Slabs.

Fig. 5.—The Pneumatic Tube-Cleaner.

filled with concrete, and the buttresses were cast. When this cast-insitu concrete had hardened twenty-four wires spaced equally up the wall were tensioned to improve the bond between the precast and cast-insitu concrete. Each vertical bar was then tensioned to 45 tons, thereby prestressing vertically the preeast and cast-insitu parts of the wall. Six wires in each of the six cables at the bottom of the wall were then tensioned, causing the bottom part of the wall to slide inwards. The remaining cables in the wall and about eighteen wires in the ring-beam were then tensioned. The remaining wires in the six cables at the bottom of the wall were not tensioned until the first part of the dome had been cast, thereby enabling the gap at the bottom of the wall to be packed and sealed in the dry.

Friction on the wires during tensioning was large. To assist in reducing the friction, soluble oil, similar to that used when machining steel, was injected into the ducts, and vibration was applied to the wire during tensioning. Water was pumped through the ducts to remove the oil before grouting. Openings in some of the ducts between the precast slabs enabled measurements to be made of the movements of the wires, and the stresses. Measurements of the deflection and vertical rotation of the ring-beam were also made at various stages of prestressing and construction. (A record of the measurements and the conclusions to be drawn therefrom are to be the subject of a report by the Cement and Concrete Association.)

The dome was cast on steel shutters supported by scaffolding on the floor of the reservoir. Concrete in buckets was lifted by crane and discharged into a skip running on a monorail above the dome.

The reservoir is constructed for the Royal Borough of New Windsor; the Borough Engineer and Surveyor is Mr. G. S. Baker. The consulting engineers are Messrs. Sandford, Fawcett & Partners. The general contractors are Messrs. Holst & Co., Ltd. The circumferential prestressing was by means of equipment supplied by P.S.C. Equipment, Ltd. The vertical prestressing was by the Lee-McCall system.

Reinforced Concrete Association.

A NEW class of membership, termed Professional Members, of the Reinforced Concrete Association was formed recently, and is open to firms and persons of standing who practise as architects, consulting engineers, or quantity surveyors. The new class is additional to the present Industrial membership, which includes civil engineering and building contractors, and manufacturers and suppliers of precast products and materials for concrete, and Associate membership, which comprises individuals engaged in the design or construction of reinforced concrete and who are in the employ of member firms, public bodies, or Government departments or who are in the teaching profession.

Particulars of the new class are obtainable from the Secretary of the Association, Mr. A. B. Harman, at 94 Petty France, London, S.W.1.

Testing of Model Structures.

THE model structures in *Figs.* 1 to 6 are some of those described at the symposium on Models for Structural Design organised by the Cement and Concrete Association and held in London last year. The model in *Fig.* 1 was made and tested at the Instituto Sperimentale Modelli e Strutture, at Bergamo, Italy, and those in *Figs.* 2 to 6 at the Association's laboratories at Wexham Springs, Bucks.

The model (to one-fourteenth scale) shown in *Fig.* 2 was tested in connection with a bridge to be built over the River Medway and comprising end spans 300 ft. long, two cantilevers each 200 ft. long and a central suspended span 100 ft. long. The consulting engineers are Messrs. Freeman, Fox & Partners. (See page 287.)

A perspex model, to one-fiftieth scale, of a balanced cantilever of Clifton bridge, Nottingham, is shown in *Fig.* 3. This prestressed structure, which is described in this journal for June, 1958, is a skew bridge and comprises two side spans of

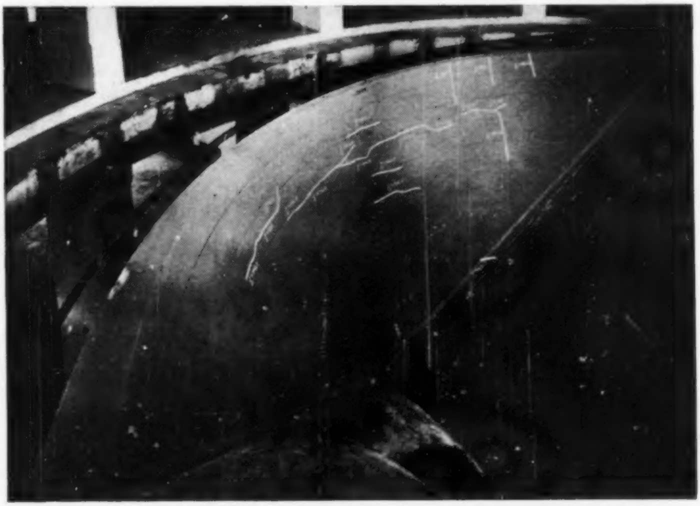


Fig. 1.—Model of a Dam, Showing Cracks after Test.

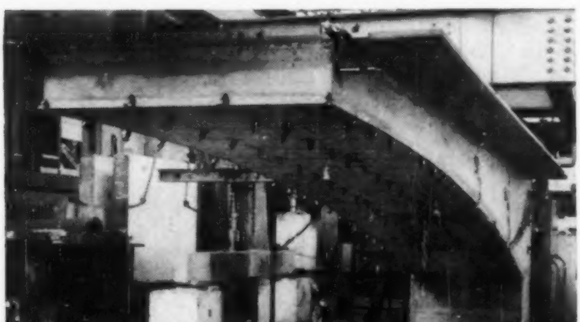


Fig. 2.—Model of a Cantilevered Beam for the Medway Bridge.

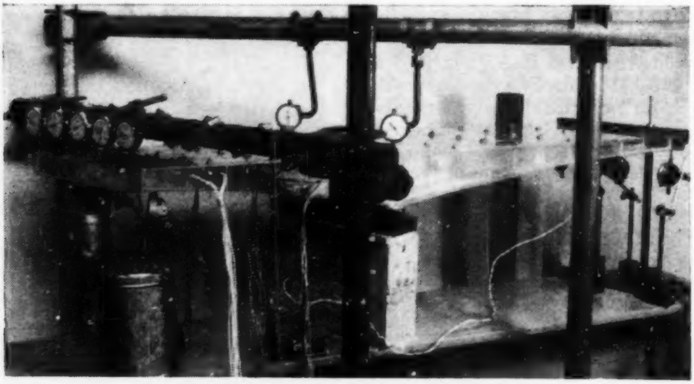


Fig. 3.—Model of a Balanced Cantilever.

125 ft. each and a central span of 275 ft. The test was undertaken to determine the "shear centre" of the abutment end for loads in various positions, the general distribution of the load due to the skew, and the effect of the eccentric prestressing forces on the deformation of the cantilever. The consulting engineers are Messrs. R. Travers Morgan & Partners.

The model shown in *Fig.* 4 is part of the roof of a garage at Lincoln, for which the consulting engineer is Dr. K. Hajnal-Kónyi. The roof comprises four hyperbolic-paraboloidal slabs $2\frac{1}{2}$ in. thick, each of which is 47 ft. 6 in. square in plan. The lower corners of the slabs are connected by a prestressed tie-beam cast monolithically with the slab. The test was made to determine the effect of the tie-beam on the stresses in the slab, and

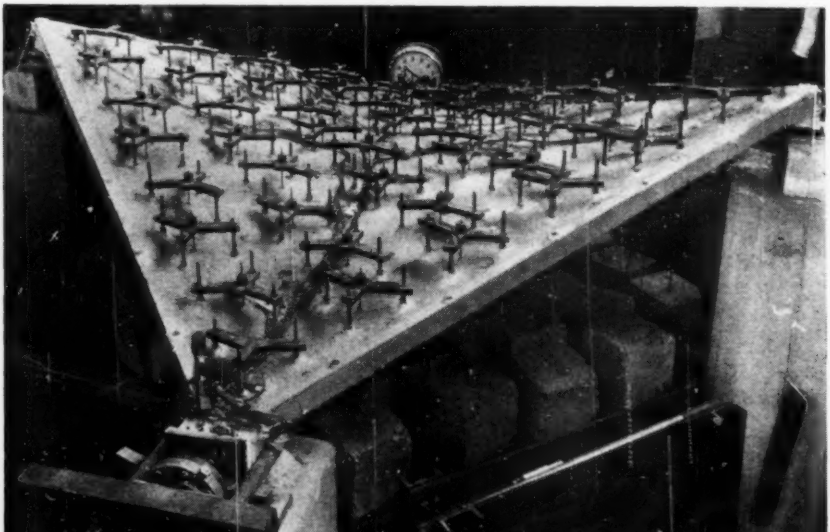


Fig. 4.—Model of Part of a Hyperbolic-paraboloidal Roof.

the stresses due to the imposed load. The model is one-tenth actual size. The structure is described in this journal for January, 1960.

Fig. 5 is a model of the deck of a slab bridge with Ministry of Transport abnormal load in position. Tests on this and similar models were undertaken to establish a theory for the calculation of transverse bending moments.

Fig. 6 shows a model of a roof for a factory at Bedford which comprises twenty-nine slabs each 48 ft. square. The test was undertaken, on a model to a scale of one-sixth, primarily to determine the deflection of the slab under uniform load and under non-uniform load due to snow, but information relating to the stresses in the edge beams and slab was also obtained. The consulting engineers are Messrs. Oscar Faber & Partners. It is expected to give a description of this structure in a future number of this journal.

Fig. 7 shows a model constructed to a scale of ¼ in. to one foot of a new railway under-line prestressed concrete bridge at London Road station, Manchester. The structure is about 300 ft. long and comprises a main central part in the form of a box-girder supporting a passenger platform, footbridge, luggage lift, and buildings; the railway tracks are supported on slabs cantilevered from the central part. To simulate the final prestressed concrete structure, the model was supported on four points, and deflection tests were then made. The model is 6 ft. long and was cast in an epoxy resin by Messrs. McKinlay Electrical Manufacturing Co., Ltd., for Mr. A. N. Butland, the Chief Civil Engineer of the London Midland Region, British Railways.

Model Dams.

Laboratório Nacional de Engenharia Civil of the Ministry of Public Works of Portugal has published recently four booklets dealing with tests on models of concrete dams and the interpretation of the results of such tests. These are: Technical Paper No. 128, "A Method of Quantitative Interpretation of the Results Obtained in the Observation of Dams", by M. Rocha, J. L. Serafim, and A. F. da Silveira (price 10 escudos). Technical Paper No. 129, "Observation of Concrete Dams. Results Obtained in Cabril Dam", by M. Rocha, J. L. Serafim, A. F. da Silveira, and M. Q. Guerreiro (price 37.50 escudos). Technical Paper No. 130, "Model Tests and Observation of Bouçã Dam", by M. Rocha, J. L. Serafim, A. F. da Silveira, and M. E.

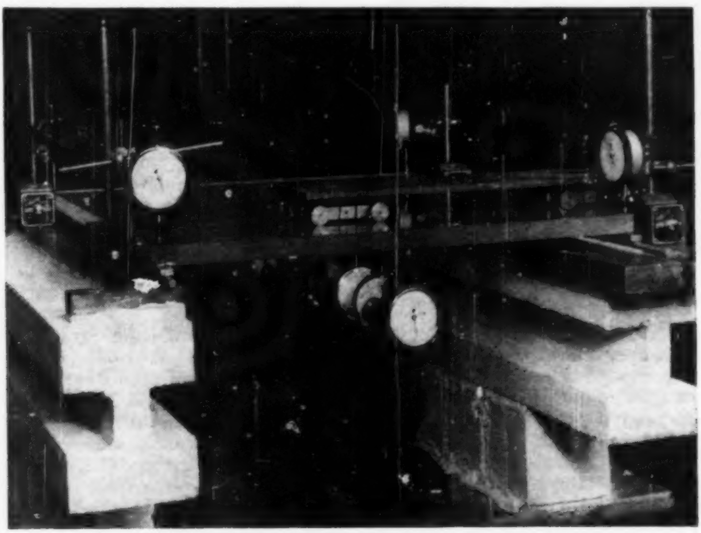


Fig. 5.—Model of Deck of a Slab Bridge.

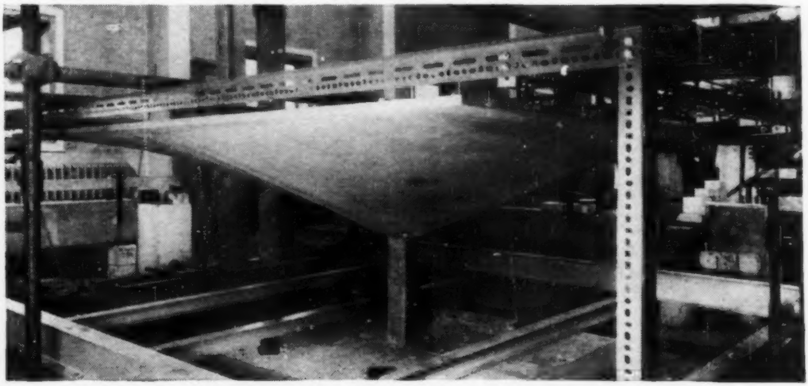


Fig. 6.—Model of Complete Hyperbolic-paraboloidal Slab.

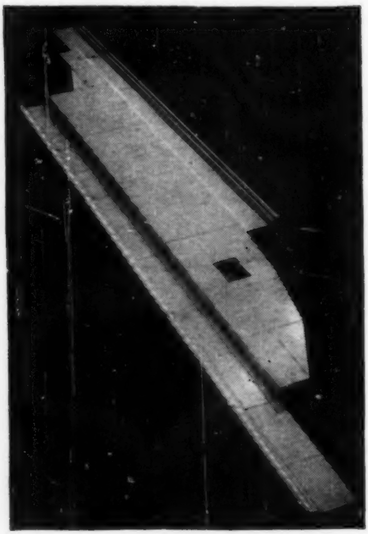


Fig. 7.—Model of Railway Bridge.

Campos e Matos (price 15 escudos). Technical Paper No. 133, "Determination of Thermal Stresses in Arch Dams by Means of Models", by M. Rocha and J. L. Serafim (no price stated).

The booklets, which are reprints of papers presented at the sixth Congress on Large Dams held in New York in 1958, are in the English language and are obtainable from the Laboratório Nacional de Engenharia Civil, Lisbon.

Model Tests in U.S.A.

Papers dealing with testing model structures and presented at the annual convention of the American Concrete Institute were as follows.

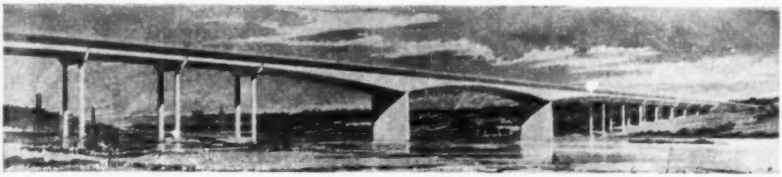
"Tests of a Rigid-Frame Bridge Model to Ultimate Load." By Ching-Sheng Wu, Arpad S. Papp and D. H. Pletta.—A one-tenth scale model of an existing skewed reinforced concrete rigid-frame bridge was tested within the elastic range and up to initial cracking of the concrete and thence to ultimate load. The model was of concrete reinforced with $\frac{1}{8}$-in. deformed bars.

"Structural Models Evaluate Behavior of Concrete Dams." By J. M. Raphael.—An unusual arch-and-buttress structure was proposed for a dam at Oroville, 750 ft. high and 5000 ft. long. A plaster-celite one-two hundredth-scale model was cast in glass-fibre moulds to represent the central portion of the dam. Live-load tests were simulated by rubber bags under pneumatic pressure. Dead-load stresses were determined at various stages of construction.

"Glen Canyon Dam Structural Model Tests." By G. C. Rouse.—Model tests were made of an arch dam 700 ft. high. The model was of plaster-celite to a scale of one to two hundred and forty. To satisfy a condition assumed in the structural analysis, the modulus of elasticity of the material from which the foundation and abutments were formed was one-sixth of the modulus of elasticity of the material of the model dam.

New Bridge over River Medway.

LARGEST PRESTRESSED CONCRETE SPAN.



THE illustration shows the proposed new 21-span bridge over the River Medway near Rochester. It will be a prestressed concrete structure 3270 ft. long comprising a western approach viaduct of eleven spans, an eastern approach viaduct of seven spans, and three river spans, the central span of which will be 500 ft. long and will therefore be the largest in the world in prestressed concrete. (Other large spans include one of 403 ft. built in 1954 over the river Moselle at Coblenz, and one of 375 ft. built in 1953 over the river Rhine at Worms.)

The new bridge, which is in connection with the Medway Towns motorway, will provide for two carriageways, cycle tracks and footpaths, and will be constructed on the cantilever principle to cause least possible obstruction to navigation. The two main piers of the bridge will be hollow reinforced concrete shafts on plain concrete foundations, constructed within steel sheet-pile cofferdams which will be taken down about 50 ft. below high-water to reach the underlying chalk. The superstructure will be two independent bridges, the side spans being extended into the main span by cantilevers 200 ft. long, which will support the 100-ft. suspended central span. The overall depth of the girders will be about 35 ft. at the river piers and 8 ft. at the middle of the central span. The anchored and cantilevered spans will be of cellular construction. The suspended span of each of the two bridges will comprise four precast prestressed concrete beams each weighing up to 95 tons. The deck will be a reinforced concrete slab cast in position. The approach viaducts will comprise reinforced concrete slabs cast on precast prestressed concrete beams each weighing up to 185 tons and supported on reinforced concrete framed piers founded on piles or on spread footings. Accurate assessments will be made of the deflections due to the dead load during construction, particularly in the case of the main spans. Shrinking and creep of the concrete and the weight of the plant and erection equipment will be taken into account to ensure that the final levels are correct and that the necessary movement for variations of temperature can take place. A model used as a basis for the design of the cantilevers is described on page 283.

The work will take about two-and-a-half years and will cost about £2,325,000. The contractors are Messrs. J. L. Keir & Co., Ltd., in association with Messrs. Christiani & Nielson, Ltd. The consulting engineers are Messrs. Freeman, Fox & Partners.

Incentive Schemes in the Building Industry.

A PAMPHLET of sixteen pages entitled "Incentives in Building", which has been issued by the Building Research Station, summarises the result of an investigation into the working of incentive schemes, and is an abstract of a full report published in 1959. ("Incentives in the Building Industry." National Building Studies Special Report No. 28. Published for the Department of Scientific and Industrial Research by H.M.S.O.) Copies of the pamphlet are available free on application to The Director, Building Research Station, Garston, Watford, Herts.

The contents include brief information on the following subjects: the percentage of saving which should be paid; forms of target-unit used; how a job is divided into operations; how schemes affect earnings, productivity, and quality; operating a scheme. The recommendations on the working of a scheme should be of value.

Book Reviews.

"**Simplified Design of Reinforced Concrete.**" By H. Parker. (London: Chapman & Hall, Ltd. Second edition. 1960. Price 52*s.*)

THE previous edition of this book on reinforced concrete practice in the U.S.A. was published seventeen years ago. Therefore considerable revision has been necessary, and much new matter, such as a chapter on prestressed concrete, is included. A feature of the book is the numerous worked examples of design calculations which are in accordance with the latest American codes. The contents include a few pages on materials and making concrete, and several chapters on bending and shearing which are followed by sections dealing with the principles of the design of rectangular and flanged beams, floors, columns, foundations, retaining walls, and stairs. This book of about three hundred pages is a good introductory text for those requiring a knowledge of the elements of reinforced concrete design as practised in America.

"**Kempe's Engineer's Year Book for 1960.**" (London: Morgan Brothers (Publishers) Ltd. 65th edition. Two volumes. Price 87*s.* 6*d.*)

THIS well-established work of reference has been revised, as in previous years, to include recent advances in science and practice of engineering. In this edition there are about 120 pages of new matter, most of which deals with subjects not necessarily within the orbit of civil and structural engineers. The section on prestressed concrete was in print before the publication of British Standard Code No. 115, but in general the recommendations of the draft of this Code are given. Likewise, regarding the section on reinforced concrete, British Standard Code No. 2007 dealing with liquid-containing structures was issued after the publication of this year-book.

"**Elementare Schalenstatik.**" By A. Pflüger. (Berlin: Springer-Verlag. Third edition. 1960. Price 19.50 D.M.)

THE third edition of this book on the elementary theory of the design of shell roofs does not differ from the previous edition except for a few small revisions. The analyses are based on the membrane theory. Formulæ for various cases of spherical, conical and cylindrical shells and barrel-vaults are given in an appendix. The book explains the principles of a theory which is generally considered to be difficult, and furnishes the basis for understanding the more difficult specialist publications and for undertaking some of the simple problems met in practice.

"**Cemento Armato Precompresso: Teoria, Esperienze, Applicazioni.**" By Carlo Cestelli-Guidi. (Milan: Ulrico Hoepli. Fourth edition. 1960. Price 6500 lira.)

THE theory and application of prestressed concrete as practised in Italy is described comprehensively (in the Italian language) in this well-illustrated book of seven hundred large pages. Statically-determinate and statically-indeterminate structures are dealt with. The examples of the application of prestressed concrete are taken from many countries and, as would be expected, some of the works of Professor Nervi are included. The Italian regulations issued in 1960 for prestressed concrete are printed in full with extensive explanatory notes by the author.

"**Bruchsicherheit bei Vorspannung ohne Verbund.**" By H. Rüsch, K. Kordina, and C. Zelger. (Berlin: Wilhelm Ernst & Sohn. 1959. Price 10 D.M.)

THIS brochure of the Deutscher Ausschuss für Stahlbeton describes tests to determine the validity of the statement made in the German code of practice DIN 4227, that the difference between the stresses in the steel at failure of a prestressed beam with post-tensioned unbonded steel and at working load (after most of the creep and shrinking have taken place) is about 20,000 lb. per square inch if the stress at failure does not exceed the yield stress of the steel. The tests showed a deviation from plus 9 per cent. to minus 15 per cent. of the prestressing force at failure. The differences based on the working load were very much greater and depended on the magnitude of the moment due to the load. Notwithstanding these discrepancies, the empirical formula is recommended for approximate calculations. Diagrams based on the results of the tests are given to aid more accurate analyses.

Book Received.

"**Road Builder.**" By Eric Leyland. "Men of Action Series," No. 6. (London: Edmund Ward (Publisher) Ltd. 1960. 88 pages. Price 9*s.* 6*d.*)

Reconditioning Thirlmere Aqueduct.

THE aqueduct, in which water is conveyed a distance of about a hundred miles from Lake Thirlmere, Cumberland, to Manchester, was constructed in 1894 and comprises forty-five miles of pipe-line, fourteen miles in tunnel, and thirty-seven miles of cut-and-cover construction. The aqueduct supplies normally 50,000,000 gallons of water daily, the rate of flow being about 2 ft. per second.

Reconditioning is now necessary and is being carried out in the cut-and-cover section, which is about 7 ft. square and has an arched roof. Water is being lost through defective parts of the aqueduct and there is an uncontrollable ingress of surface water in some parts. Remedial works, which were commenced in 1958, require taking sections, in excess of $\frac{1}{4}$ mile in length, out of service by pumping the water through a by-pass pipe of 30 in. diameter laid on the ground, thus enabling the section of the aqueduct to be completely emptied and the inner faces of the walls exposed for examination, testing and repair where necessary.

The procedure differs in the northern and southern sections of the aqueduct. In the former, due to the fact that much more faulty concrete exists, the entire wall of the aqueduct is cut with a modified coal-cutting machine, and the walls are then refaced. In the southern section, less deterioration was found and less rigorous treatment is necessary. The work comprises initial cleaning and inspection, pressure grouting, cutting-out and making-good where necessary and further grouting if required. Some variation in the method of treating the defective parts is practised. Where the decomposition of the concrete is fairly deep, the affected part is cut out and a tube inserted in the space created should this space penetrate the wall. The purpose of the tube is to relieve the water pressure outside the wall while initial sealing by means of a quick-setting compound is made. As soon as a seal has been effected, high-alumina cement may be used to strengthen the seal, the remainder of the repair being of 2 : 1 : 1 concrete. The tube can be used for further grouting if necessary and is plugged on completion.

While reconditioning is in progress, the floor is being strengthened by the laying of a 4-in. to 6-in. slab of 4 : 2 : 1 concrete. Conveyance through the aqueduct is by means of small electric-battery trucks.

The work is being carried out by Manchester Corporation using direct labour mainly because the unknown nature and the extent of the repairs made tendering uncertain. The Engineer and Manager of the Manchester Corporation Waterworks is Mr. W. K. Lewis, B.Sc., M.I.C.E., M.I.Mun.E., M.I.W.E., to whom we are indebted for the following description of the defective concrete.

In 1902 it was found that a considerable quantity of water was leaking from a short section of the aqueduct where it crosses through and around a range of carboniferous-limestone hills near Hutton Roof in southern Westmorland. It was the practice to crush local stone for aggregate and here the limestone had been used. Upon investigation it was found that the peaty acidic water from Lake Thirlmere had attacked the aggregate, resulting in leaks into the surrounding rock, the water disappearing underground without trace. The remedial measures taken at that time were to reline the walls and invert with a 1 : 1 sand-cement mortar to protect the aggregate from further attack. These works are described by M. R. Barnett, M.I.C.E., in the Journal of the Institution of Civil Engineers, March 1907, in a paper entitled "Repairing a Limestone-Concrete Aqueduct".

On other sections of the aqueduct which had not been thoroughly inspected or repaired since constructed in 1894, the main points where leakage or inflow of water occurred were in the invert or in the lower 18 in. of the walls. When the aqueduct was constructed, the walls were built first without projecting footings, a flat invert then being placed between the walls. No special precautions were taken other than roughening the concrete at the foot of the walls to seal the joint between walls and the invert. Most of the leaking appeared to take place at this joint. No displacement of the invert appeared to have been caused by upward pressure as the provision of drains and pressure-relief valves was adequate, but

cracks at construction joints had widened in some cases to allow further leaking. Mushy concrete varying in depth up to the full thickness of the wall occurred only in spots and these could be accounted for by the concrete being insufficiently mixed or indifferently placed, and therefore liable subsequently to attack from flowing water through the honeycombed surface. Possibly small lumps of clay fell unnoticed from the sides of the trench and were mixed with the concrete behind the wall shutters. For the most part, concrete surfaces in contact with water were softened to a depth of $\frac{1}{8}$ in. to $\frac{1}{4}$ in. only. The arched roof, except for isolated spots, was in perfect condition; the level of the water flowing in the aqueduct was usually no higher than springing of the arched roof.

The New Kingsferry Bridge.

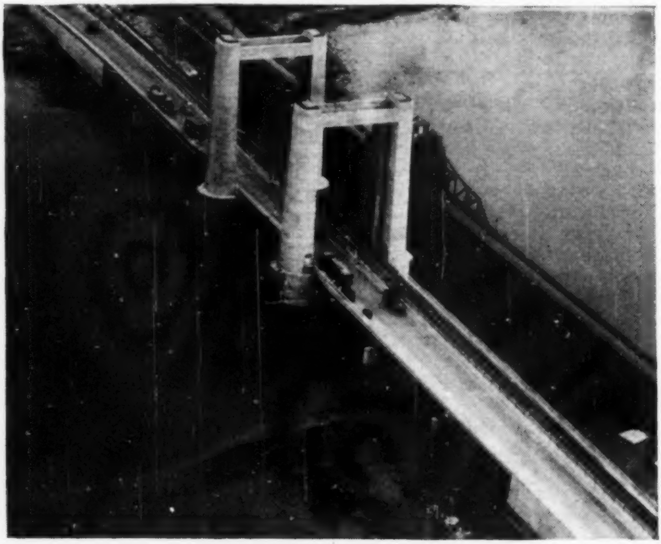


Fig. 1.

A FEATURE of the new Kingsferry Bridge (*Fig.* 1) between the Isle of Sheppey and the mainland of Kent is the twin towers which are 130 ft. high and are of reinforced concrete. The bridge is otherwise of steel construction with a central lifting-span of 90 ft. length. This type of bridge is uncommon in Great Britain. The lifting-span, which weighs 465 tons, rises vertically between the two pairs of towers to give a clear opening 95 ft. high at high tide. The machinery chambers at each main pier are supported on two concrete cylinders which are founded on London clay at about 60 ft. below high-water level. The cylinders are hollow and of 32 ft. external diameter with reinforced concrete walls 5 ft. 6 in. thick. Between the piers there is a service tunnel of 8 ft. 6 in. internal diameter.

On each side of the lifting-span there are three side spans each about 80 ft. long. Each pier of the approach spans is supported on five reinforced concrete piles 4 ft. in diameter founded on London clay at about 80 ft. below high-water level.

The deck of the bridge is 50 ft. wide and accommodates a single-line of railway, a 24-ft. carriageway, and a 6-ft. footpath. The bridge, which was opened in April last, is 650 ft. long and cost about £1,381,000. It was built by the Kent County Council. The consulting engineers were Messrs. Mott, Hay & Anderson, and the general contractors were Messrs. John Howard & Co., Ltd.

Terminology of Prestressed Concrete.

WITH reference to our recent Editorial Note and subsequent correspondence on this subject, the following letter has been received from MR. A. J. HARRIS, B.Sc. (Eng.), M.I.C.E.

" Perhaps you will allow me, if not to justify, at least to explain the words pretensioned, post-tensioned and tendon on which your strictures intrigued me. I am well placed to do so; I claim to have invented them.

It was when I was in Paris working with M. Freyssinet, in about 1948. The word 'prestress' was already in use, being the direct translation of the French 'précontrainte', a word introduced just before the war. Its meaning was and is clear—the application of permanent stresses to a structure prior to loading; in the particular case of prestressed concrete, the prestress can be compressive, tensile, shearing, torsional, or any combination of these four, although compression usually predominates. A word was needed to distinguish between two sorts of prestressed concrete, that where wires are tensioned first and concrete cast around them and that where concrete is cast and allowed to set and the wires then tensioned; to make this distinction, the word post-stressed was creeping in, a word which, in view of the above definition, is meaningless; hence the words pre-tensioned and post-tensioned. Their precise use would be in some such phrase as prestressed concrete with pre-tensioned steel, shortened for convenience to pre-tensioned prestressed concrete; an impure locution, perhaps, though the figures of speech known as ellipsis and transferred epithet have long been classic in the English language and we need not be so pure as to make utterance impossible. I agree with you, Mr. Editor, however, in finding 'pretensioned concrete' a form of shorthand so extreme as not to convey any meaning.

As for tendon, the need was similar. Bars, bundles of wires, strands and single wires were being either used or proposed for use and a generic word would be useful. I hesitated between tendon and ligament but felt that tendon was in sense and in sound nearer to the idea of that part of a prestressed structure which is tensioned.

It is rare that an engineer is in a position to name things; having named them, he finds that there may be a difference between what the thing is, what the name means, and what the name is used for. A knowledge of the first two should enable so literate a journal as your own not to slip up on the last."

A Church in the Dominican Republic.

THE accompanying illustration shows the Higuey Basilica, which is a church dedicated to Nuestra Senora de la Altagracia and recently completed in the Dominican Republic. The church will accommodate three thousand persons and the covered cloisters will accommodate many thousands of pilgrims to this shrine. The entire building is of reinforced concrete, and the main parabolic arch forming the tower is 240 ft. high. It is stated by the contractors that there was considerable difficulty in casting in place the towers and arched roofs. The cost of the work was 3,500,000 dollars, excluding the cost of the internal finishes. The designers are Dunoyer de Segonzac and Pierre Dupre. The contractors are Concretera Dominicana, C. por A.

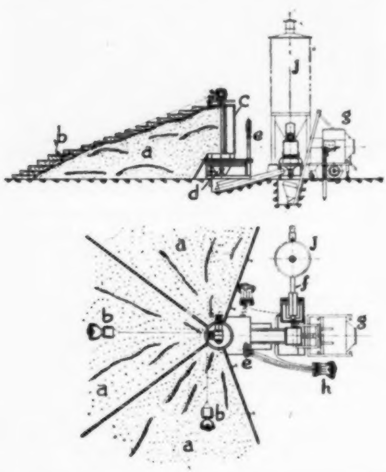
A Batching Plant at Ground Level.

THE batching plant in *Fig.* 1, which has been recently introduced into Great Britain from Germany, is operated by two men. It includes an aggregate store comprising up to six vertical partitions built radially around a central steel pillar above a weighing hopper, a motor-operated scraper guided by hand, and a screw conveyor from a cement silo with an automatic device for measuring the cement.

The aggregate is banked against the central pillar by the scraper. At the base of the central pillar in each storage area is an outlet which is opened and closed by a horizontal shutter operated by means of a flexible cable connected to a hand-lever near the weighing scale. The outlets permit aggregate to fall into the weighing hopper which is at ground level. The accumulative weight of each addition of aggregate of each size is indicated on a scale. The cement is delivered by the screw conveyor to a separate weighing hopper and when the required amount has been delivered the conveyor stops automatically. The cement and aggregates are discharged on to a short belt-conveyor and delivered to a skip in which they are raised and fed into the mixer. One man guides the scraper and the other operates the weighing hopper. The plant is claimed to be able to batch the material for 26 cu. yd. of concrete per hour.

There are several variations of this plant. For example, the belt-conveyor can be omitted if a deeper pit is excavated below the hopper so that the materials can be discharged directly into the skip of the elevator; the output by this arrangement may be about 20 cu. yd. per hour. For larger outputs, a jib from which the scraper is operated is provided and the operator then controls the scraper from a cabin above the central pillar. It is possible in this case for the batching to be automatic and controlled by the same operator, and the hourly output may be up to 59 cu. yd.

A plant is also available in which the batching of the materials, and the charging, mixing, and discharging of the concrete are fully automatic and a man is required only to control the scraper. The hourly output in this case may be up to 65 cu. yd. The plant is obtainable in Great Britain from A.C.E. Machinery, Ltd.



a.—stored aggregate; *b.*—scraper; *c.*—central pillar; *d.*—hopper; *e.*—scale; *f.*—cement conveyor; *g.*—mixer; *h.*—control pillar; *j.*—cement silo.

Fig. 1.

A Contribution for Research.

MESSRS. McCall & Co. (Sheffield), Ltd., have offered a sum of money annually to the Highway and Traffic Engineering Post-graduate School of the University of Birmingham to encourage research on reinforced and prestressed concrete roads and bridges. For 1960 the money will be used to purchase books for the library. In subsequent years part of the money will be used to keep the library up-to-date and part will be allocated as a prize for the most outstanding paper or project on concrete roads or highway structures.

Bulletin Received.

"Structural Interaction of Walls and Floor Slabs. Effect of Deformation of Joints on Load-carrying Capacity of Brick Masonry Walls, Lightweight Cellular Concrete Element Walls, and Over-reinforced Concrete Frames." By Sven Sahlin. (Stockholm: 1959. Royal Institute of Technology. Bulletin No. 33. No price stated.)

The Constructional Industries.

One person in sixteen is employed in the constructional industries of Great Britain and nearly half of the nation's capital investment is in construction. The total expenditures on construction, including maintenance and repairs, in 1959 were £2,394,000,000 in Great Britain and £120,000,000 oversea by British firms. These facts are given in a booklet entitled "Statistics on Construction", published in May 1960 by the Federation of Civil Engineering Contractors (price 1*s.* from the Federation, Romney House, Tufton Street, London, S.W.1). The recent increase in industrial construction follows three years in which work had been falling off at the rate of £60,000,000 annually. The decreasing amount of industrial construction work, together with a reduction of £56,000,000 in two years in the orders for public works, accounts mainly for the present surplus capacity in the civil engineering industry. There is disappointment at the slowness of the road construction programme as a consequence of which only half of the road-making capacity of the industry is employed. Construction costs fell two points last year, and are now 13 per cent. above those of 1954, in spite of an increase of 27 per cent. in wages since that date. Construction output rose by 6 per cent. in 1959 and the labour force increased by up to 2 per cent.

Production of Sand and Gravel.

The quantities of building sand, concreting sand, and gravel produced in various parts of Great Britain in 1959 are given by the Ministry of Works in a leaflet entitled "Sand and Gravel Production, 1958–59" (H.M.S.O., price 1*s.* 3*d.*). The total quantity of aggregate produced in 1959 was 55,000,000 cu. yd., which represents an increase of 12 per cent. on the amount of 49,000,000 cu. yd. produced in 1958. Increases exceeding 30 per cent. were recorded in Kent and East Sussex, and parts of East Anglia. Particulars of production are given for each of the regions and service areas defined by the Advisory Committee on Sand and Gravel. The production in each county is also given for each of the years from 1954 to 1959. The greatest increase in any county was in Bedfordshire, where the production in 1959 was nearly two-thirds more than in 1958. There were small reductions in Hertfordshire, Norfolk, Northumberland and Worcestershire.

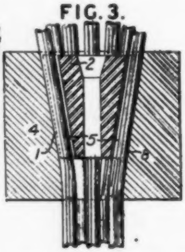
Patent Applications.

Anchors for Prestressing Cables.

An anchorage device for a bundle formed by a predetermined number of wires in a frusto-conical cavity is constituted by a number of keys equal to that of the wires and in the form of at least longitudinally tapered wedges each of which is positioned between two adjacent wires, the keys being carried on the periphery of a deformable frusto-conical body and two adjacent keys carried by the body providing an outwardly-flared wire housing at the periphery of the body, into which housing each wire can pass only partly in order that the wires do not contact the bottom of the housing and project beyond the external face of the keys. As shown, wedges (1), preferably of steel, are carried on a deformable frustum (2) of a cone which may be of rubber and wires (5) of a

FIG. 1.

FIG. 3.



bundle to be anchored are engaged between the wedges and project partially from between them into engagement with the inner frusto-conical surface (8) of a member (4). The wedges are partially embedded in the frustum (2) as shown, or where the frustum (2) is constituted by a thin tube may be attached to the tube by welding. In another arrangement the wedges are formed integrally with the frustum which may be a casting or stamping of a metal having a high elastic limit and capable of withstanding substantial plastic deformation before fracture. The outer member (4) may be formed from a solid metal block or, in an alternative construction, may comprise a tubular casing, an inner metal jacket, and a filling of concrete. The frustum (2) may be constructed of cement mortar which is sufficiently deformable to allow relatively small displacement of the wedges. In another arrangement some of the wedges may be slidable along the frustum. The wedges may be grooved or corrugated.—No. 819,828. Soc. Technique Pour L'Utilisation de la Précontrainte S.T.U.P. Procédés Freyssinet. January 23, 1957.

Tensioning Reinforcing Wires.

Before tensioning a reinforcement wire 14 extending through concrete 21, a thrust plate 15, a gripper sleeve 17, a pressure tube 18 and a disc 19 are arranged about the wire. A tensioning jack (*see* Group I) carrying an axially movable ring 13 is then positioned above the wire with the wire engaged by the jaws of the jack. During tensioning of the wire by the jack

FIG. 8.

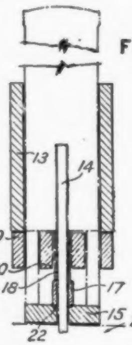
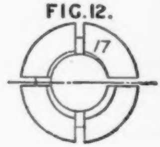


FIG. 12.



the tube 18 is pressed by the jack on to the gripper sleeve which enters the depression 22 in the disc and is caused to grip the wire. When the required wire tension is reached the disc 19 is struck by the ring 13, a tapered bore 20 in the disc causing the sleeve to increase the grip on the wire. The jack, pressure tube and disc are removed from the wire, the free end of the wire is cut off, and the remaining end of the wire is burred over the upper end of the sleeve.—No. 821,075. Speedacc Co., Ltd. January 15, 1958.

Bridge Girders Prestressed with Large Strands.

The girders of the bridge described on page 233 of our number for June 1960 were prestressed by the Gifford–Udall large-strand system. This structure is the first in this country to incorporate post-tensioned strand of large diameter.

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